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ROCK TUNNELING

with

STEEL SUPPORTS

by

ROBERT V. PROCTOR, M. E.*
Former Vice President and General Manager
Commercial Shearing, Inc.

THOMAS L. WHITE, P.E.
Registered Civil Engineer
Fellow, American Society of Mechanical Engineers
Chief Consulting Engineer, Retired
Commercial Shearing, Inc.

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with an

Introduction to Tunnel Geology

by

KARL TERZAGHI*
Member, American Society of Civil Engineers
Member, Institute of Civil Engineers, London
Consulting Engineer

BIBLIOTECA CENTRAL
SISTEMA DE BIBLIOTECAS
PONTIFICIA U.C. DE CHILE

*Deceased

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FOREWORD

The need for tunnels has increased greatly in the past few decades as expansion of population and productivity has demanded better and faster transportation for people, water, fuels, sewage and other services. These tunnels pass through hills, under bodies of water, and populated areas to avoid interference from natural barriers and surface movement. Tunnels obviously require excavation of ground or rock. This book deals principally with tunnels in rock.

When excavation is made for a tunnel it is generally necessary to install temporary support to hold the surrounding material until a permanent lining can be placed. In earlier days these supports were almost always made of timber since wood was the only material available and it was plentiful.

Steel supports were sporadically used in rock tunnels in the first quarter of this century, usually patterned after the timber sets which were commonly used at that time. In the late 1920's and early 1930's, steel began to be adopted more widely as its superiority was gradually recognized. The increased use of steel supports in rock tunnels was probably due to the almost universal use of metallic linings in earth tunnels. Steel supports were, and still are, designed and fabricated to utilize the strength of steel to full advantage.

Steel support is available to meet any requirement of rock tunnel construction. It is usually designed to fulfill the specific requirements of each job, therefore components may take numerous forms. Each has its special advantages to meet the needs of any tunnel cross section or condition, or to take care of some particular phase of the tunnel builder's problems.

It is the purpose of this book to briefly review problems which involve support systems and suggest means of solving them to the end that rock tunneling can be done faster, cheaper and safer.

The subject is covered in four sections:

First—A section devoted to engineering geology contains information which is required for estimating rock pressure on tunnel supports. This section was prepared by Dr. Karl Terzaghi whose works constitute outstanding authority on the subject. Dr. Terzaghi, prior to his death in 1963, was a member of the American Society of Civil Engineers and the Institute of Civil Engineers (London).

For more than forty years he was connected in a consulting capacity with many projects involving difficult foundations and tunnels throughout the world. He authored more than two hundred technical papers dealing with various aspects of his professional activities, and for many years lectured on Engineering Geology at the Graduate School of Engineering, Harvard University, Cambridge, Mass. In the section he prepared for this book he explains rock behavior and gives a succinct outline of what a tunnel builder should know about rocks and rock pressure in order to protect himself against serious miscalculations. Special attention is given to the type and accuracy of the practical information which can be derived from results of a geological survey prior to construction of the tunnel.

Second—A section devoted to the relationship between rock behavior, type of steel support, and the method of excavation.

Third—Analytical methods of design of the supporting structure under assumed loading conditions are discussed.

Fourth—A catalogue is included containing descriptions of steel supports manufactured by Commercial Shearing, Inc. Designs and details of these supports are based on experience gained in supplying steel supports to many hundreds of tunnels for almost a half century.

Thus, the book is intended to be helpful to the designing engineer for the project, the resident engineer on the job, the contractor, his project engineer, and his superintendent.

In reviewing the text prior to a new printing of this volume, a number of figures have been changed using more recent jobs to illustrate the text, and where necessary, data brought up to date. Values in the design section have not been adjusted to reflect higher allowable stresses since the procedures are merely explanatory and finite values are not required here. Safe load tables were adjusted in a previous issue to comply with A-36 steel specifications.

Mr. Robert V. Proctor, for many years Vice President and General Manager of Commercial Shearing, Inc., whose perseverence was largely responsible for this book, passed away in 1967. By bringing designs and cost data to contractors and engineers, Mr. Proctor did much to show the construction industry the merits of steel tunnel support. He also traveled widely in connection with tunnel supports as a representative of his company.

Being the sole survivor of the trio which prepared the original text, I feel a responsibility for keeping the book up to date. There is also a compelling desire to let the readers know that the three co-authors always felt immense and personal satisfaction that their efforts in setting down these data from personal experience and studies have been of value to the construction industry. It is my sincere hope that this revised volume will continue to serve this great industry.

THOMAS L. WHITE

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SECTION I

ROCK DEFECTS AND LOADS

ON

TUNNEL SUPPORTS

By

KARL TERZAGHI

INTRODUCTION

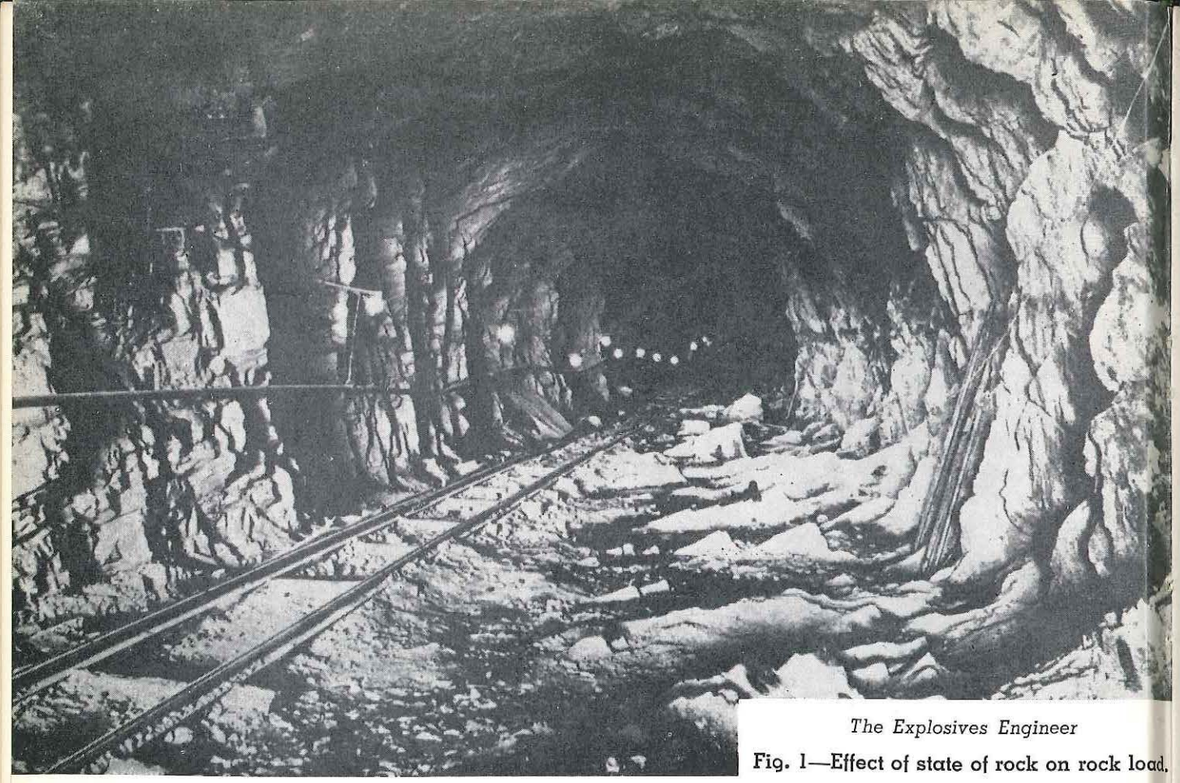
Among tunnel builders the term "rock" has always been applied to natural aggregate of mineral particles which required the use of explosives in tunneling. In recent years boring machines have been developed for cutting various materials including soft and medium rocks, the design of cutting tools depending on hardness of the material to be penetrated. The costs and hazards of tunneling through rock depend primarily on the condition of the rock which may be intact, broken, or decomposed.

The condition of a given kind of rock may change so greatly within the length of a single tunnel that the pressure which it exerts on the tunnel support may vary between zero and 10 to 15 tons per square foot. This well known fact is illustrated by *Figures 1 a* and *1 b* which show two sections of the Moffat Tunnel in Colorado. Both tunnel sections are located in a rock known as gneiss. Yet one section, located in intact gneiss, is unsupported, whereas the other, located in crushed and decomposed gneiss, required heavy timbering.

The chief causes of rock pressure are mechanical defects, such as fractures, and chemical defects due to the decomposition of individual mineral constituents of the rock. The nature of these defects and their manifestations in the tunnel are very different for different rocks. Therefore, an elementary knowledge of the nature and common defects of the principal types of rocks is a prerequisite for adequate tunnel design, and it is useful to the engineer engaged in tunnel construction.

The following chapters contain first of all a brief review of the principal types of rocks and their specific properties. Next comes a discussion of their most common mechanical and chemical defects which have a direct bearing on tunneling. The results of the discussion furnish the basis for an inquiry into the sources and the intensity of the load which acts upon the artificial support of the walls of tunnel in broken, crushed and decomposed rock.

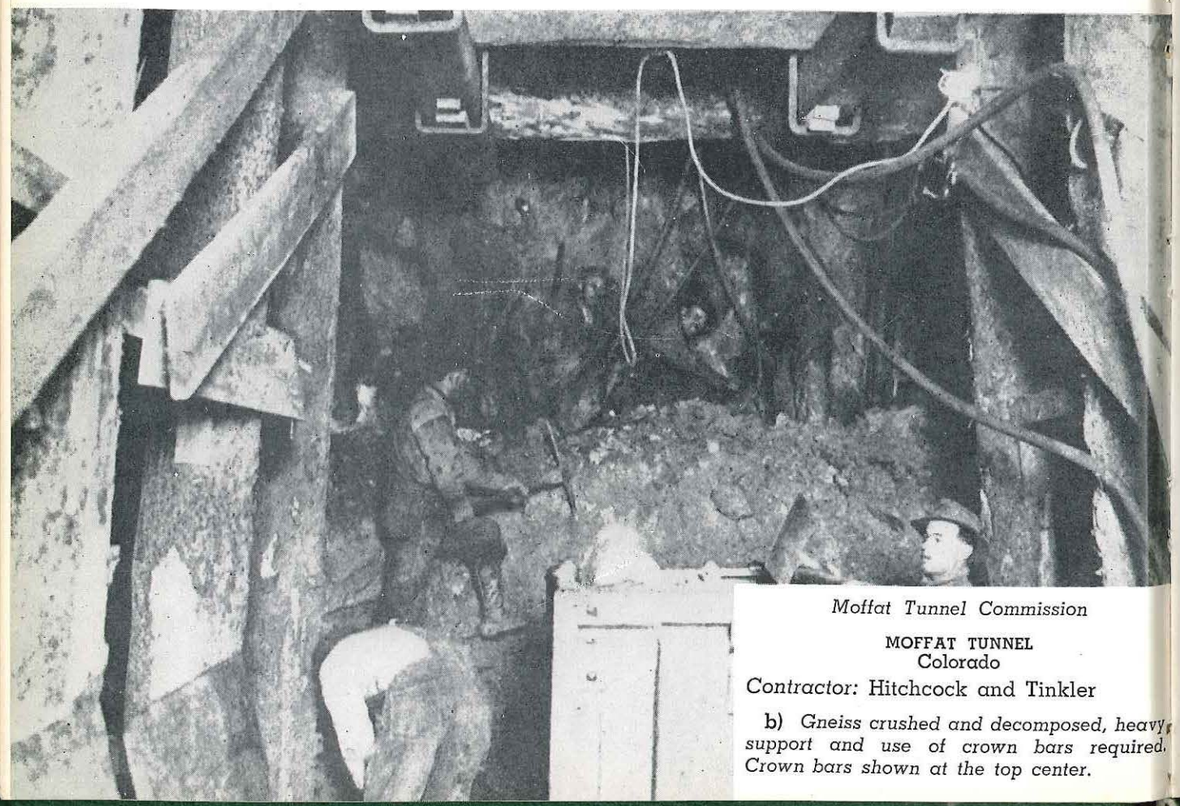
The review concludes with a resume of the procedure for estimating the rock load and a few references to readily accessible sources of useful information.



The Explosives Engineer

Fig. 1—Effect of state of rock on rock load.

a) *Gneiss intact, no support required.*



Moffat Tunnel Commission

MOFFAT TUNNEL
Colorado

Contractor: Hitchcock and Tinkler

b) *Gneiss crushed and decomposed, heavy support and use of crown bars required. Crown bars shown at the top center.*

CHAPTER 1

PRINCIPAL TYPES OF ROCKS

MAIN CATEGORIES

The rocks which are likely to be encountered in tunnels can be divided into three large groups: *igneous, sedimentary* and *metamorphic rocks*. The origin of the members of these groups can be compared to that of the three principal construction materials: steel, concrete, and brick.

IGNEOUS ROCKS

One class of igneous rocks, described as *extrusive*, emerged from the interior of the earth in a molten state through vents or fissures in the uppermost part of the earth's crust which is cool and solid. Upon emerging, the molten rock flowed over the landscape, forming streams or lakes of lava. Owing to relatively rapid cooling the resulting rock is fine-grained. Two common representatives of this group are rhyolite and basalt.

Rhyolite is a light-colored rock, chiefly composed of quartz and feldspar. Its unit weight is about 165 lbs. per cu. ft. *Basalt* is a dark-colored rock composed chiefly of feldspar and pyroxene. Its unit weight may be as high as 180 lbs. per cu. ft. Dark-colored fine-grained igneous rocks, such as basalt or andesite, are commonly called *trap rock* by engineers.

Rhyolite flows commonly do not cover areas greater than a few hundred square miles, whereas basalt flows have inundated regions which are hundreds of thousands of square miles in extent. In the northwestern United States several successive eruptions of basalt flooded an area of more than 250,000 square miles and the maximum total thickness of the flows exceeds 4,000 feet.

In some localities the molten rock never reached the surface of the earth; it remained within the earth's crust in chambers, which it produced either by doming the solid rock above it or by stoping and by melting the solid rock. In these chambers the molten rock cooled and crystallized very slowly. As a result of slow cooling, the constituents occur in the form of crystalline grains large enough to be seen with the unaided eye.

Rocks which originated in this manner are known as *intrusive* rocks. *Granite* is a common representative of this group. Its chemical and mineralogical composition is similar to that of rhyolite, from which it is distinguished by its coarser grain. *Diorite gabbro* is another member of the group; its composition corresponds approximately to that of basalt.

SEDIMENTARY ROCKS

Principal types of sedimentary rocks

Sedimentary rocks may be divided into several groups. The most important group comprises the clastic sedimentary rocks. The term *clastic* is applied to rocks which are composed of fragments produced by the disintegration of previously existing rocks of any kind. The clastic sedimentary rocks include *conglomerate, sandstone,* and *shale.* Another group of sedimentary rocks includes most *limestones* and *dolomites,* which consist largely of the hard parts of such marine organisms as clams and corals.

Origin and deposition of sediments

The raw material of the clastic sedimentary rocks consists of rock waste. The debris produced by rock weathering is picked up by brooks, transported in suspension by rivers or shore currents and deposited at the end of the line of transportation, in a lake, on a flood plain, or in the proximity of a sea shore. The coarse grained sediments are known as *sand* and *gravel* and the fine grained ones as *silt* and *clay*. By the deposition of cementing materials in their interstices, sand and gravel are gradually transformed into sandstone and conglomerate.

The chief ingredients of limestones and dolomites originated *in situ*, but they are always mixed with a certain amount of sand, silt, or clay which was transported and deposited by shore currents.

The velocity of the currents which transport the raw material of sedimentary rocks varies with the seasons. At low velocity the water can carry very small particles only, whereas at high velocity coarse particles can be carried in suspension. Therefore, sedimentary rocks are likely to be stratified. Layers of sandstone or limestone are commonly separated from each other by thin layers of *shale*, and beds of coarse grained sandstone may alternate with beds of fine grained sandstone.

The boundaries between the successive layers are known as *bedding planes*. In their original position the bedding planes are usually more or less horizontal. The thickness of the individual layers may range between a fraction of an inch and many feet.

Cementation of coarse grained sediments

Gravel and sand are commonly transformed into rock by the precipitation of cementing material from an aqueous solution. Under appropriate conditions, such precipitation can take place very near the surface. This process has taken place in many recent valley fills in California, and has transformed loose sand and gravel to soft rock in which tunneling is relatively easy. The cementing material which converted the cohesionless accumulation of shell fragments into limestone commonly originated in local solution and redeposition of calcium carbonate.

Consolidation of fine grained sediments

The rocks which are formed by the consolidation of fine grained sediments such as silt or clay are known as *shales*. The transformation takes place not by cementation as it does in coarse grained sediments, but by local intergrowth between adjoining grains. This intergrowth is associated with at least slight mineralogical changes. It does not take place unless the sediment is subjected to at least moderately high temperature and pressure acting over a considerable period of time. Otherwise it remains soft indefinitely, like some of the clay strata which cover vast areas in northeastern Russia. These strata are almost as old as the oldest known sedimentary rocks.

The transition from silt and clay to shale at depth is gradual. Therefore in tunnels materials may be encountered which are intermediate between typical silt or clay and typical shale. These may be described as *petrographic halfbreeds*. They combine all the undesirable characteristics of the parent material with those of a rock, and their mechanical properties may vary within an extremely wide range. The mineralogical changes associated with the transformation of a silt or clay into shale are disclosed by the fact that the unit weight of the solid matter of shale in thick strata commonly increases with increasing depth below the surface. The product of the transformation should not be called a shale unless it possesses two properties: when struck with a

hammer it should give a clear ring, and when immersed in water its volume should remain unchanged. Any shale-like material which does not exhibit these properties must be classified as a compacted sediment and not as a rock.

Shale formation and crustal movements

The increase in temperature and pressure required to transform silt or clay into shale can develop only if the parent material becomes at least temporarily buried beneath a layer of sediments with a thickness of at least several thousand feet. Yet, both silt and clay are commonly shallow water deposits. Therefore the reader may

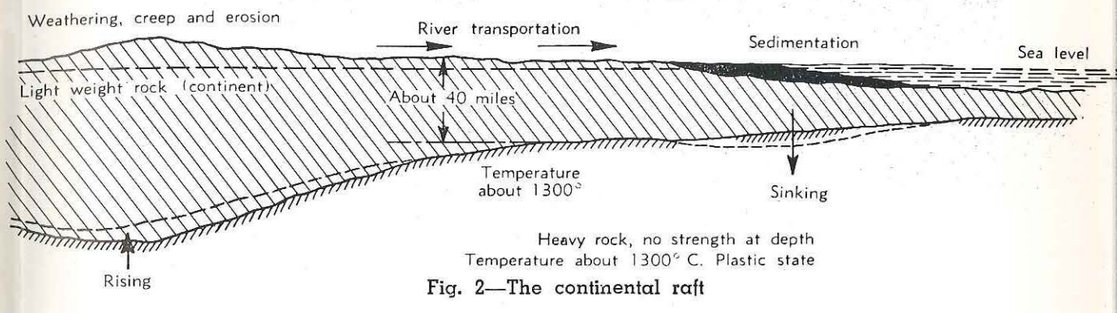


Fig. 2—The continental raft

inquire how it is possible that strata with such an origin may gradually sink to a depth where they are buried beneath a very thick overburden. The geologist's conception of this process is illustrated by *Fig. 2*. The continents can be compared to flexible rafts which float on an ocean composed of rocks with a slightly higher unit weight. The base of these rafts is located at a depth of roughly 40 miles.

Every miner knows that the temperature of the earth increases with increasing depth below the surface. At the base of the continental "rafts" the temperature of the rocks is about 1300° C. At such high temperatures every rock flows under the influence of any state of stress other than that which exists in a stationary liquid. In such a liquid the pressure acts at any given point with equal intensity in every direction. The flow is extremely slow, much slower than the flow of hard asphalt under the weight of a pebble resting on its surface. But the flow due to a non-hydrostatic state of stress does not stop until hydrostatic equilibrium is reestablished.

In *Fig. 2* the left-hand side represents a zone of erosion on the elevated part of the surface of a continental "raft," the middle part is a zone of transportation and the right-hand part a zone of sedimentation. If weight is removed from one part of a floating flexible raft and shifted to another, the relieved part rises and the loaded one sinks. Therefore, on the continents, the zones of erosion go up and those of sedimentation subside. This concept is based on the combined results of innumerable gravity determinations and of accurate surveys of the gradual subsidence of coasts in the vicinity of deltas.

Subsidence under load is commonly associated with subsidence due to other still unknown causes. An important subsidence of this kind has occurred in the region of the Mississippi River delta. Oil well records from this region indicate that the delta is underlain by about 30,000 ft. of shallow water deposits.

METAMORPHIC ROCKS

Origin of metamorphic rocks

Metamorphic rocks represent the result of a process of recrystallization which took place at high temperature and high pressure. The properties of the product depend on

the nature of the rock subject to metamorphosis and, to a considerable extent, on the deformation associated with the process.

Changes produced by high temperature and pressure

When a rock is subjected for a long time to high temperature combined with high pressure, its porosity decreases, its strength increases and the unit weight of the solid material commonly increases due to the loss of chemically bound water. Limestones are transformed into *marble* and sandstones into *quartzite*. Under the influence of moderate temperature and pressure, shales are transformed into fissile *slates* and *schists*, which are known as *low-grade metamorphic rocks*. With increasing temperature and pressure, they are metamorphosed to very hard and dense *gneiss*, classed as a *high-grade metamorphic rock*. Since the metamorphic rocks derived from silt and clay are among the most obnoxious troublemakers in tunnel construction, they deserve the tunnel builder's special attention.

Properties of slates and schists

Nearly all slates and schists were derived from shales. During metamorphism of a shale, the percentage of micaceous minerals becomes greater, the average size of the particles increases, and practically all of the flaky particles become oriented parallel to one plane. As a result of this parallel orientation of particles, slates and schists split very readily into thin plates or slabs. The capacity to split in this manner is known as *cleavage*. Cleavage is most perfectly developed in *roofing slate*. The presence of cleavage weakens the rock mechanically to such an extent that its compressive strength in the direction of the cleavage planes is very small. Whenever the rock undergoes deformation under low confining pressure, near the surface of the ground or at the walls of a tunnel, it breaks into thin plates. The moisture which gradually accumulates in the open fissures gives rise to chemical alteration known as *weathering* which further weakens the rock. *Fig. 3* shows beds of a clay sediment which were folded and transformed into a low-grade metamorphic rock. It should be noted that the cleavage is roughly parallel to the axial plane of the fold. The pressure which transformed the sediment into a low-grade metamorphic rock acted at right angles to the cleavage.

Some metamorphic schists contain thin, smooth sheets of quartz oriented parallel to the cleavage planes. If these sheets are fairly continuous, the process of drilling in such schists can be extremely difficult and expensive.

Properties of gneiss

If pressure and temperature are high enough and act long enough, schist may be transformed into *gneiss*. In gneiss, as in schist, the individual crystals are oriented approximately parallel to a plane or a warped surface, but the percentage of flaky constituents is much smaller than in schist. Therefore, the cleavage is very imperfect and the strength of gneiss in an intact state may be equal to that of an igneous rock. It represents the last stage in the transformation of clay sediments into solid rock.

The mineralogical composition of gneiss is generally similar to that of a granite. If gneiss consists of the same constituents as granite and its cleavage is poorly developed, it may be called a *granite-gneiss*. In a tunnel, granite-gneiss breaks much like granite as shown in *Fig. 1 a*.

Cleavage surfaces in gneiss are commonly very uneven and wavy. Cleavage surfaces with such charactertistics can be seen on the roof of the tunnel shown in *Fig. 22*.

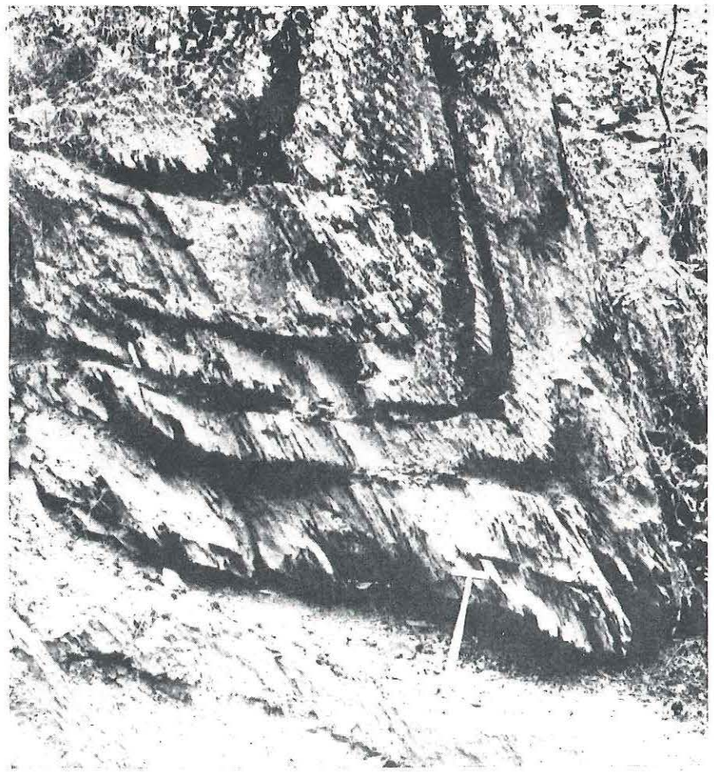


Fig. 3—Slaty cleavage

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The cleavage planes cut across folded beds of slate. The pressure which produced both the folds and the cleavage acted at right angles to the cleavage.

CHAPTER 2

MECHANICAL DEFECTS OF ROCKS

PRINCIPAL TYPES OF MECHANICAL DEFECTS

Every rock, without exception, has more or less conspicuous mechanical defects which have no direct connection with its inherent properties. They consist of more or less closely spaced fractures. Simple fractures are known as *joints*. Fractures of large extent along which a relative displacement of the adjoining masses of rock have occurred are called *faults*. In some instances the rock adjacent to faults is completely crushed. Such rock constitutes a *crushed zone*.

If a rock has innate mechanical defects such as bedding or cleavage planes, the joints and faults constitute a supplementary source of weakness.

JOINTS

Definition and origin of joints

The term joint indicates a crack or a fracture in a rock along which no noticeable displacement has occurred. A joint can be open or closed. Closed joints may be nearly invisible. Yet they constitute surfaces along which there is no resistance against separation. In quarries the spacing of joints determines the largest size of blocks of sound rock which can be obtained. Therefore joints and joint systems have attracted the attention of builders ever since cut stones have been used.

Joints in many igneous rocks are due to the volume contraction associated with cooling. Many of the joints in deformed rocks of any kind are due to failure by tension. The origin of the joints in undeformed sedimentary rocks such as limestone or sandstone is not yet clearly understood. However, it can be taken for granted that almost every rock contains joints.

Joints in igneous rocks

In igneous rocks which cooled rapidly the joints are generally closely spaced. A well-known example is *columnar basalt*, which consists of columns oriented at right angles to the surface of cooling. The columns commonly measure from five to ten inches across. Since the joints between the columns are open, water circulates freely through them. In contrast to basalt, rhyolite has a tendency to develop closely spaced and irregular joints.

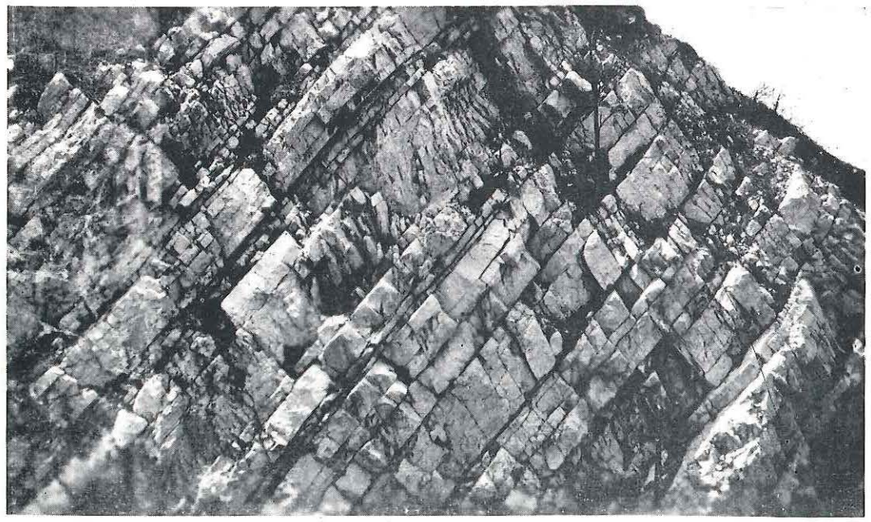
The joint system in coarse grained igneous rocks such as granite commonly consists of three sets of joints which divide the rock into more or less prismatic blocks. The width of the blocks may range between a few inches and many feet. In some parts of the country the orientation and the spacing of the joints in granite is almost constant over large areas, whereas in others it changes from place to place in an erratic manner.

In many massive rocks other than extrusive igneous rocks, the joints are either not continuous or so irregular that the blocks located between them are intimately interlocked. Hence the blocks cannot change their relative position without some fracturing along their contacts. Nevertheless the joints break the continuity of the rock and reduce the average strength of the jointed mass to a small fraction of that of the same rock in an intact state.

Joints in sedimentary and metamorphic rocks

Sedimentary rocks also commonly contain three sets of joints, one of which is invariably parallel to the bedding planes. The others commonly intersect the planes at approximately right angles.

The three sets of joints of sedimentary rocks can clearly be seen in *Figs. 4 a* and *b*. Opinions regarding the origin of the joints at right angles to the bedding planes are still controversial, but the presence of these joints in almost every rock can be taken for granted.



a) *Catskill red shale and sandstone showing the three sets of joints characteristic of sedimentary rock. (Pennsylvania Turnpike, east of Sideling Hill)*

b) *Beekmantown limestone showing similar joint system (Quarry near Tyrone, Pa.)*
Pennsylvania Geological Survey

Fig. 4—Joint systems in sedimentary rocks.

In limestone and sandstone, the joints of each set are commonly several feet apart. In shale, they are generally closer, and they may be so close that no intact specimen can be secured with a width of more than a fraction of an inch. During excavation, such shales disintegrate into small angular fragments. The surfaces of the fragments of some shales are shining and striated. Such surfaces are called *slickensides*.

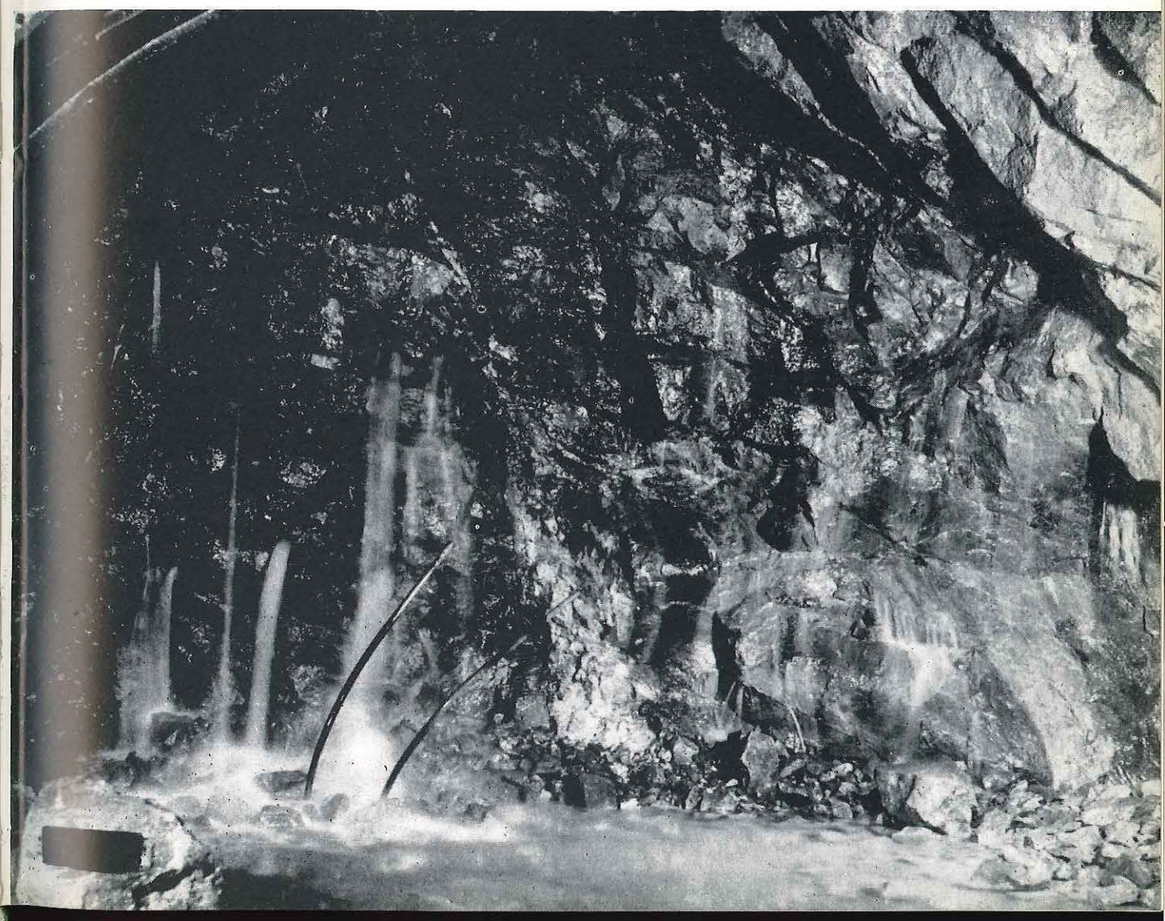
Metamorphic rocks commonly contain two or more sets of joints oriented approximately at right angles to the direction of cleavage.

General characteristics of joint systems

Since joints are among the most important causes of excessive overbreak and of trouble with water, they always deserve careful consideration. There is ample evidence that in almost every rock the spacing of the joints increases and the width of the joints decreases with increasing depth below the surface. To a depth of about 100 or 150 ft. the joints in many hard rocks, such as granite, are so wide and so numerous that

Fig. 5—Water flowing from seams and joints in granite at great depth.
The condition shown in the photograph is rather unusual, as joints at great depth are commonly so nearly closed that water merely trickles or seeps from the rock.

Metropolitan Water District of Southern California
LOS ANGELES-COLORADO RIVER AQUEDUCT, SAN JACINTO TUNNEL near Banning, California
Contractor: Force Account



almost every drilled well strikes water. Below this depth the prospects of striking a water-bearing seam decrease very rapidly.

Nevertheless, even at much greater depth, tunnels are often wet. Exceptionally, very large quantities of water may be encountered. The San Jacinto Tunnel in California is an example. Part of the western section of the tunnel, located in jointed granite, was mined from the Potrero shaft with a depth of 796 ft. In spite of the great depth the quantity of water which had to be pumped from this tunnel rose to 16,200 gal. per min. *Fig. 5* shows the water flowing out of the seams in the working face.

The general character of the joint system can usually be determined in advance of construction by a careful examination of rock exposures located in the vicinity of the tunnel line. Supplementary information can be secured by means of diamond drill holes. However, the spacing of the joints and their water-bearing capacity can hardly ever be predicted reliably.

FAULTING, FOLDING AND THRUSTING

General characteristics

The terms faulting, folding and thrusting indicate the effects of major movements in the earth's crust involving displacements along planes of failure known as *faults*, bending of strata into *folds* or both combined. In contrast to the processes which lead to jointing, the intense deformation resulting in the formation of faults and folds occur only within geographically limited districts commonly known as *zones of tectonic disturbance.*

Normal faults and reverse faults

In some regions, the earth's crust is broken up into individual strips or blocks. Each of the blocks is relatively undisturbed, but some of them have subsided with reference to their neighbors along planes which dip at a steep angle towards the block that went down. These planes are known as *normal faults*. *Fig. 6 a* illustrates a normal fault across a sedimentary formation.

The direction of the trace of a fault on a horizontal plane with reference to the true north-south line is known as the *strike* of a fault. The vertical component of the

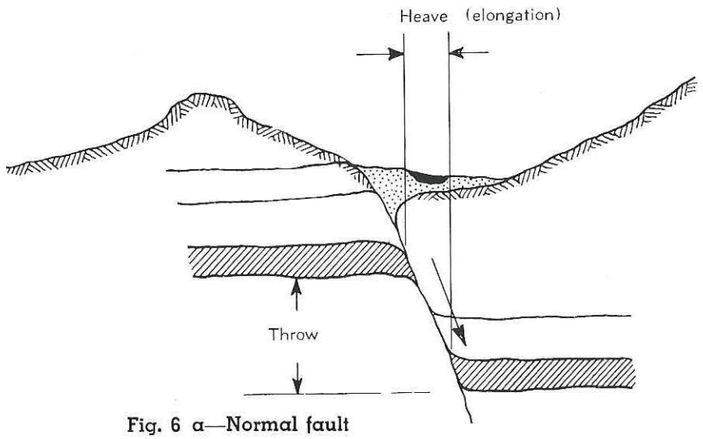


Fig. 6 a—Normal fault

displacement represents the *throw* of the fault. It may range between a fraction of an inch and several thousand feet. The horizontal component of the displacement, measured at right angles to the strike of the fault, is referred to as *heave*. It should be noted that the displacement of the masses of rock adjoining a normal fault, *Fig. 6 a*, involves an increase of the width of the area occupied by the rocks. The total elongation is equal to the heave.

Less common in undisturbed regions are the *reverse faults* illustrated by *Fig. 6 b*. These faults involve a shortening of the region, which is equal to the heave. The movement along both normal and reverse faults may be associated with a displacement in the direction of the strike of the fault. Such displacement is called *strike slip*. The strike slip of some faults is much more important than the throw. Movements of this type are chiefly encountered in folded regions which will be discussed under the next sub-heading.

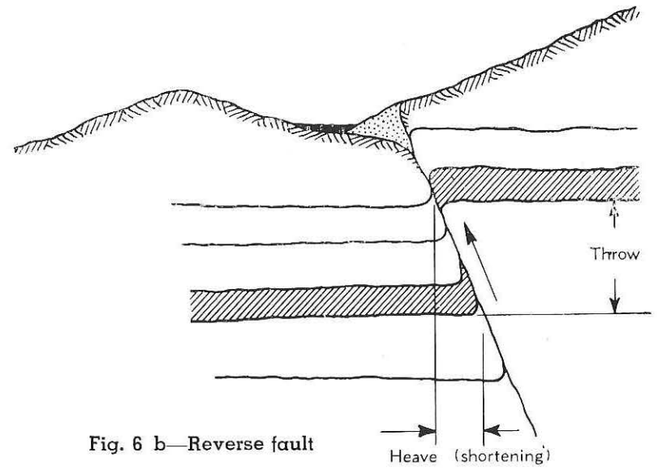


Fig. 6 b—Reverse fault

Important dislocations may occur along several more or less parallel faults, located close to each other. They constitute a *group of faults* and the zone which contains the faults is known as a *fault zone.*

Folds and thrust faults

On every continent there are several zones in which the rocks have been pressed into steep folds. As a rule the folds are more or less parallel to each other. There is one broad zone of folds near the Atlantic coast of the United States, and several others are located in the western part of the country. The forces which produced the folds were approximately horizontal and their intensity was very much greater than the intensity of the vertical pressure due to the weight of the rocks. The condition previous to the development of these horizontal pressures is shown in *Fig. 7 a*.

Gentle folds are commonly symmetrical with reference to a vertical section through the crest of the fold. Such folds are known as *symmetrical folds*. Other folds are *asymmetrical*, that is, one limb is steeper than the other, as shown in *Fig. 7 b*. Such folds are produced by a one-sided lateral thrust which may ultimately produce failure by shear along a surface rising at a small angle to the horizontal, as shown in *Fig. 7 c*. The surface of failure constitutes a *thrust fault.*

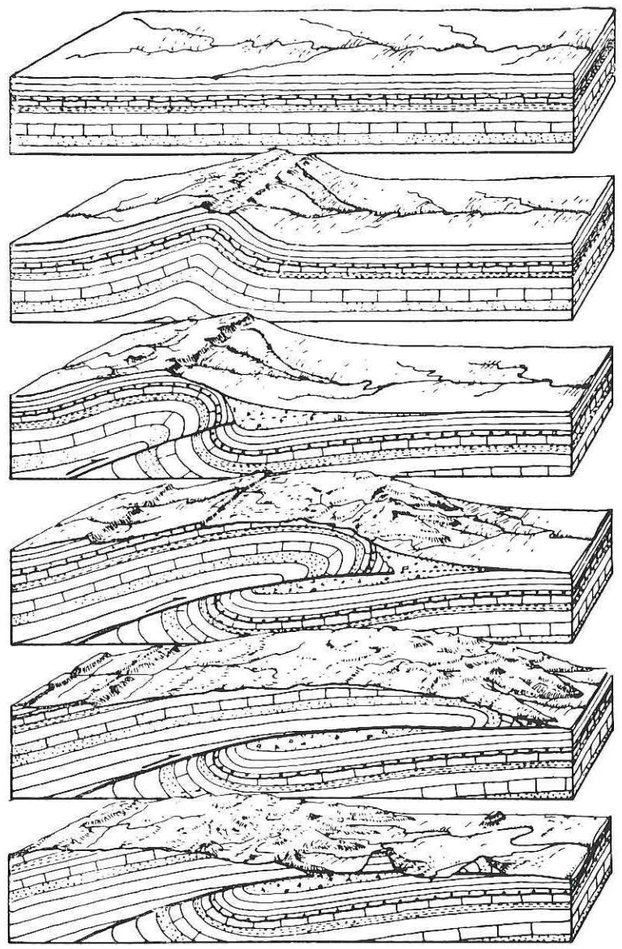


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Fig. 7—Development of an overthrust

If the horizontal pressure continues with undiminished intensity, the displacement along the fault steadily increases. In the course of this process huge masses of older rocks are shoved over younger ones as indicated in *Figs. 7 d* to *f*.

The total horizontal displacement, measured in the direction of the movement may amount to many miles. Giant mass movements of this type are known as *overthrusts*. Large overthrusts have taken place in every part of the world in which the rocks have been subjected to intense folding. In the United States these parts include most of mountainous areas of the western part of the country and the Appalachian region in the east.

The intensity of folding of a given mass of rock may range between gentle undulation and intense compression into symmetrical folds or into asymmetrical, overturned folds and overthrusts. If a zone of overthrust is approached in the direction in which the pressure acted, all types of deformations may be encountered, intermediate between those illustrated by the cross sections *b* to *f* in *Fig. 7*.

Cross faults

After a mass of rocks has been intensely deformed by folding or thrusting, it usually breaks up into blocks which are separated from each other by normal or steep reversed faults, similar to those shown in *Fig. 6*, or by faults associated with important horizontal displacements. The prevalent direction of these faults is parallel with or at right angles to the folds. Faults approximately at right angles to the folds are called *dip* or *cross faults*.

Rock defects due to faulting and folding

The space between the walls of a fault is filled with crushed and powdered rock. The thickness of the crushed zone may range between a fraction of an inch and many feet, and the crushed material may be highly permeable or almost impermeable. If the powdered rock located between the walls of a fault has a high clay content, it is called *fault gouge*. Crushed material containing a large amount of angular fragments is referred to as *fault breccia*. The rock adjoining a fault may be perfectly intact or it may be badly broken up to a considerable distance from the fault. If the rock at the walls of a fault is intact, its surface is generally polished and shiny. Such polished surfaces are known as *slickensides*.

The mechanical rock defects due to folding depend primarily on the stress-deformation characteristics of the rock. Rocks which are sufficiently strong to transmit a compressive force under given conditions are said to be *competent* under those conditions. On the other hand, rocks which are sufficiently plastic to deform without fracturing are *incompetent*. Sandstone, quartzite and igneous rocks are relatively competent under all conditions. Shale and slate are commonly incompetent. Limestone is likely to be competent at low temperature and under moderate pressure. At depths where high pressure and temperature favor recrystallization, it is likely to be relatively incompetent.

Competent strata in folded regions are likely to be intensely fractured whereas incompetent strata in the same region may be almost or entirely intact. The effect of intense deformation on a competent stratum is illustrated by *Fig. 8*. It shows the heading of a tunnel through quartzite which is a highly competent metamorphic rock composed chiefly of quartz. On account of tectonic movements the quartzite was completely crushed. The crushed material was slightly re-cemented with the result that the excavation required blasting. However, as soon as the shots were fired, the crushed and recompacted rock disintegrated into cohesionless sand.

The degree of competence of rocks also determines their condition in the proximity of large overthrust faults such as that shown in *Fig. 7 f*. If the rocks adjoining the fault are incompetent, the fault may be barely visible in the tunnel. But if some of the strata are competent, the fault is likely to be accompanied by big irregular pockets and thick layers of completely crushed and powdered rock.

Rocks consisting of minerals with relatively equidimensional crystal forms such as quartz or dolomite, have in a crushed state the properties of a sharp-grained sand. On the other hand, shales derived from clay, and schists with a high content of micaceous minerals, such as chlorite or sericite, are likely to have in a crushed state

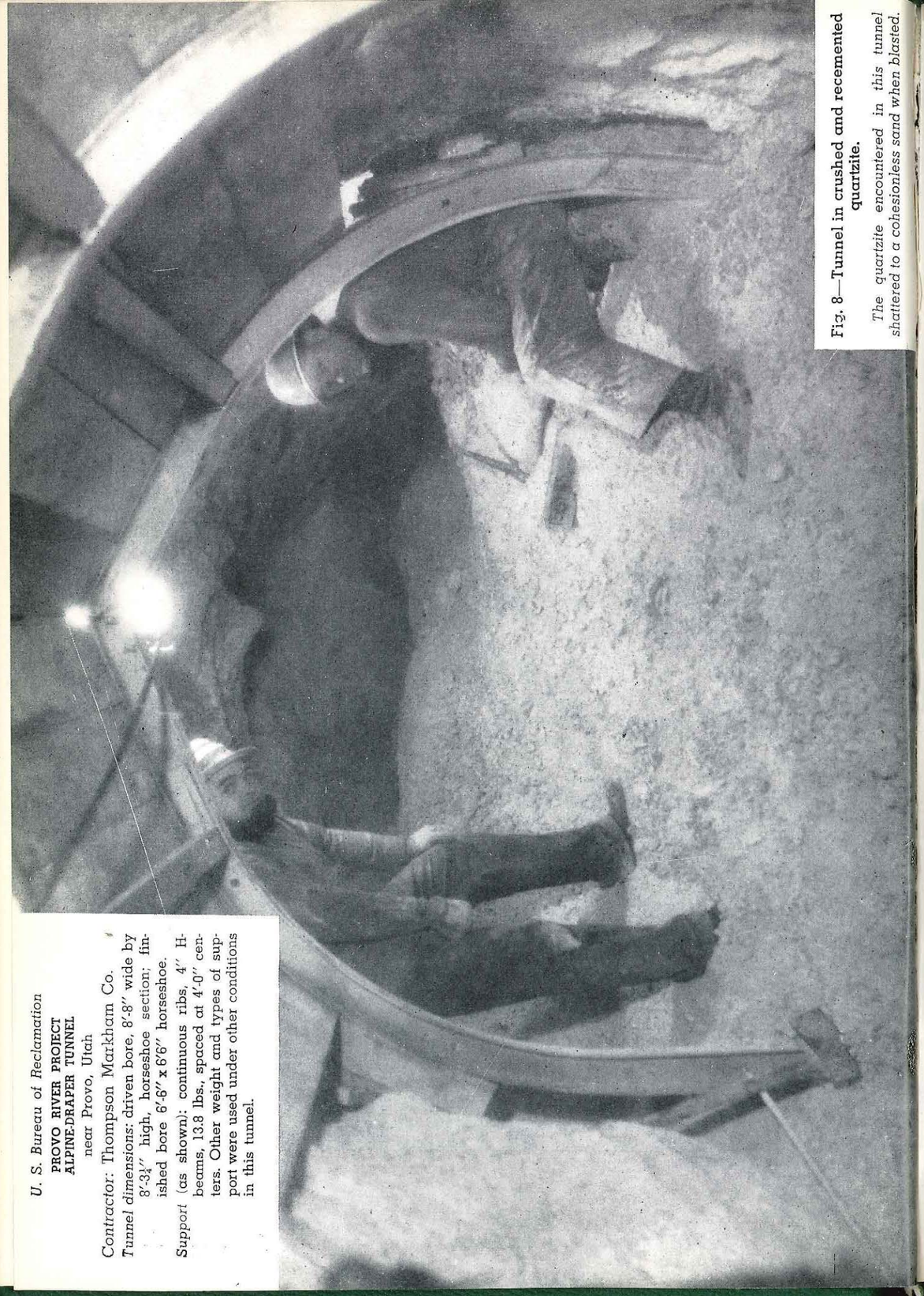


Fig. 8—Tunnel in crushed and recemented quartzite.

The quartzite encountered in this tunnel shattered to a cohesionless sand when blasted.

U. S. Bureau of Reclamation
PROVO RIVER PROJECT
ALPINE-DRAPER TUNNEL
near Provo, Utah

Contractor: Thompson Markham Co.

Tunnel dimensions: driven bore, 8'-8" wide by 8'-3¼" high, horseshoe section; finished bore 6'-6" x 6'6" horseshoe.

Support (as shown): continuous ribs, 4" H-beams, 13.8 lbs., spaced at 4'-0" centers. Other weight and types of support were used under other conditions in this tunnel.

the properties of a lean clay. This is because, in a finely divided state, these minerals have all the characteristic properties of real clay.

Healing processes in rocks

In some regions, fractures such as joints or faults have healed to such an extent that the rock is as strong as it was in an intact state. The healing process consists of the deposition of minerals on the walls of the fractures. In some rocks the healing process took place simultaneously with the opening of the fracture so that at no time was the fracture as widely open as the distance between the walls would indicate.

The substances which fill the cracks were commonly brought from great depth in hot solutions or in a gaseous state. With rather rare exceptions, as for instance the healing of cracks in limestone, the healing process seems to take place only at great depth. If, at a later stage, the rock is lifted up to the proximity of the surface, new joints and seams may be formed. Hence it is by no means uncommon to encounter in a rock both healed-up and open joints, or faults which can hardly be detected and others bordered by crushed or broken rock.

The most impressive manifestation of the healing process in rocks consists of the transformation of completely crushed rock located between the walls of overthrust faults into hard and solid rock. The cementation may even occur while the process of crushing proceeds. The resulting rock is known as *mylonite*. During tunneling through an overthrust, the rock located in the crushed zone may be encountered in any state intermediate between that of a sand or clay and that of a hard rock.

The results of the process of healing are illustrated by *Fig. 9*. The figure represents the geologist's record of his observations during tunneling through an overthrust in the northern Alps, similar to that shown in *Fig. 7 d*. The middle strip in *Fig. 9 b* rep-

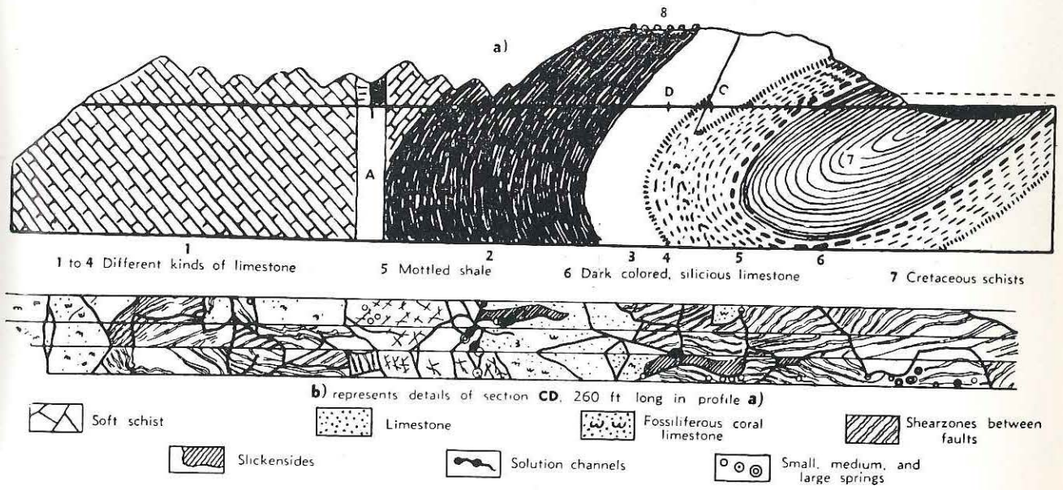


Fig. 9—Healing of fractured rock

a) represents the geological profile of a tunnel in the northern Alps. (After O. Ampferer)

b) represents the geologist's record of the rock exposures in 260 ft. of pilot tunnel from D to C in Fig. 9 a. The middle strip shows the structural details of the rock visible at the roof and the outer strips those which are visible on the side walls. Faults and shear zones were present in large numbers but were completely healed. Rock loads were very moderate and water entered only from solution channels and seams of recent origin. (After H. Ascher)

BIBLIOTECA CENTRAL
SISTEMA DE BIBLIOTECAS
PONTIFICIA U.C. DE CHILE

Board of Water Supply, New York City
DELAWARE AQUEDUCT
WEST BRANCH-KENSICO TUNNEL
North Heading from Shaft 16, near
White Plains, New York

Contractor: S. A. Healy Co.

Tunnel dimensions: driven diameter 24′-4″; finished diameter 15′-0″.

Support: full circle ribs, 8″ x 8″ WF-beams, 67 lbs., in 6 pieces spaced 4′-0″ centers.

Lagging: 6″ channels, 8.2 lbs. clamped to ribs.

Fig. 10—Tunnel face in a fault zone.

Moderately hard limestone at left side separated from jointed and partly decayed gneiss at right by a decayed crush zone in center.

Mining was by the heading and bench method with wall plate drifts. A corner of one drift is visible at right center. The planks at the left center formed the lagging for the left hand drift before the round was fired. Full circle type of steel ribs provided adequate support under conditions of heavy side pressure and unstable bottom.

resents the roof, and the adjoining strips the two sides of the pilot drift. The exposures in the drift disclosed an intricate network of faults and shear zones, and the sequence of rocks was as erratic as if the rock had been passed through a giant crusher. Yet all the faults and shear zones shown in the figure were completely healed. The rock load did not exceed that of a moderately jointed rock, and the water entered the tunnel only through solution channels and through a few seams which seemed to be of recent origin.

FAULTING AND THRUSTING IN RELATION TO TUNNELING

Effect of faults and thrusts on rock conditions in tunnel

In connection with tunnel engineering, the magnitude of the throw of a fault is irrelevant, because it is by no means uncommon that a fault with a large throw is associated with a very thin layer of gouge which can hardly be detected in the tunnel, whereas the same rock adjoining another fault with a small throw, may be badly broken over a broad belt on either side of the fault.

Fig. 10 shows a narrow fault zone which was encountered in one of the tunnels of the Delaware Aqueduct. The rock located along the fault was completely crushed within a zone several feet wide. However, the crushed material was well compacted and cohesive.

The walls of some faults are separated by a space several feet wide filled with sand or sand-like material. If a tunnel located beneath the water table encounters such a fault, a mixture of sand and water rushes into the tunnel. Such an accident occurred in the Hetch Hetchy Tunnel for the water supply of San Francisco. It caused considerable delay and expense. *Fig. 11* shows a similar flow of sand out of an open seam into one of the tunnels on the Pennsylvania Turnpike.

A geologist is generally able to predict, on the basis of the results of a geological survey, whether important faults are likely to be encountered and to indicate the approximate location of most of them. But he is rarely in a position to predict the width of the zone affected by faulting and the conditions of the rock within this zone. Hence, if faults are to be expected, local deviations from the average pressure conditions are possible but their importance cannot reliably be predicted.

Rondout Tunnel

The Rondout Pressure Tunnel of the Catskill Water Supply of New York City is an example of a tunnel through moderately folded and faulted strata. The tunnel has a total length of about 4-1/2 miles. It is located at a depth of about 400 ft. below the deepest part of the valley floor. It intersects several steep reverse faults and one normal fault and the surrounding rock consists chiefly of limestone, shale, sandstone and conglomerate. The principal difficulties were due to water and gas. A detailed description of the conditions encountered in this tunnel has been written by L. White[1].

Astoria Tunnel

The difficulties experienced in driving the Astoria Tunnel are typical of those likely to be encountered in the proximity of a big overthrust fault of the type illustrated by *Fig. 7 f.* This tunnel, with a length of about 4,600 ft., is located at a depth of about 200 ft. below the level of the East River between Astoria and the Bronx in Greater New York. At the site of the tunnel a mass of gneiss has been shoved over the younger dolomite along an uneven, gently inclined thrust fault.

1. Lazarus White, "The Catskill Aqueduct," John Wiley & Sons, Inc., New York, 1913.

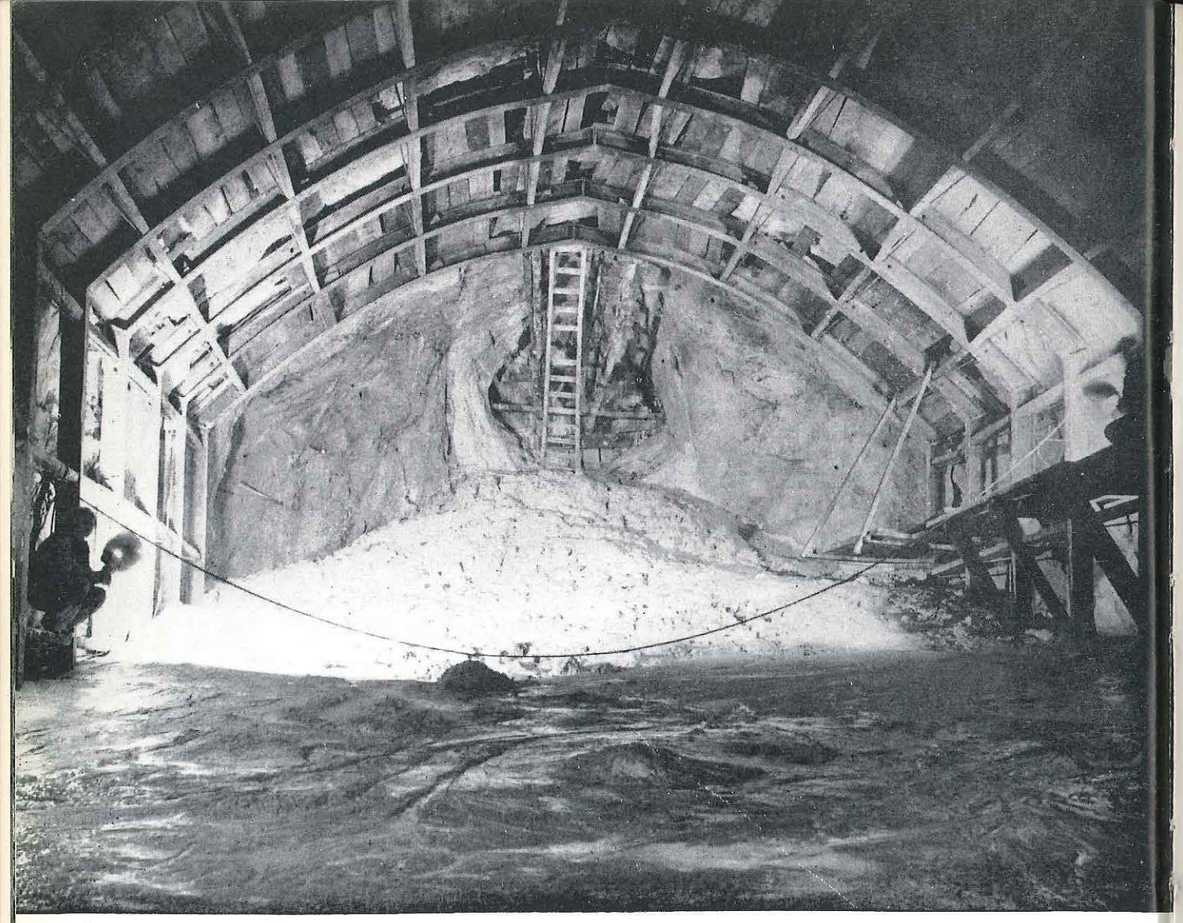


Fig. 11—Flow of sand and water from a fault.

The fault was encountered in quartzitic sandstone. The sand which flowed out of the fault accumulated at the foot of the working face as shown in the photograph.

Pennsylvania Turnpike Commission
PENNSYLVANIA TURNPIKE, KITTATINNY TUNNEL — NO. 1
West Heading, near Carlisle, Pa.

Contractor: Bates & Rogers Construction Corp.

Tunnel dimensions: driven bore, 31'-6" wide by 25'-5" high, straight side.

Support: rib and post. Ribs 8" WF-beams, 31 lbs.; posts, 8" x 6½" WF-beams, 24 lbs.; spacing varied but generally 4 ft. or 5 ft. was used.

In the eastern part of the tunnel the contact between dolomite and gneiss is fairly tight and the adjoining rock is sound. But in the western part, over a length of about 400 feet, shear zones were encountered in which the rock is reduced to a powder resembling a micaceous green sand and the adjoining rock is badly shattered. In six weeks about 1700 cubic yards of powdered and weathered rock was washed by springs out of the crushed zones into the tunnel, and the greatest inflow of water into the tunnel amounted to about 10,000 gallons per minute. The method for mining through the crushed zone has been described by J. V. Davies[1].

Simplon Tunnel

A short section of the Simplon Tunnel in Switzerland is also located in the proximity of a big thrust fault. As a consequence, a considerable part of the tunnel had to be driven through squeezing ground. In some sections, the pressure was so intense that the walls of the tunnel had to be provided with a skin-tight lining of heavy I-beams. The interstices between the beam webs were filled with quick-setting concrete.

Harlem River Tunnel

If a tunnel is located below the water table, even ordinary cross faults can be the source of considerable difficulties and loss of capital. *Fig. 12* is a section through the Harlem River along the Harlem River Siphon of the New Croton Aqueduct for the water supply of New York. The tunnel intersects a cross fault. The existence of the fault was known prior to the construction of the tunnel, but the condition of the rock on either side of the fault was unknown.

In order to avoid unnecessary trouble, the fault zone was explored prior to designing the tunnel by means of a considerable number of inclined diamond drill holes and an exploratory drift located at a depth of about 120 feet below the deepest part of the river bed. The construction of the drift was abandoned on account of unfavorable pressure and water conditions before the heading reached the fault. Investigation showed that the rock on either side of the uppermost part of the fault is badly broken and decomposed. Hence, tunneling across this zone would be very hazardous. However, with increasing depth the width of the defective zone decreases and below a depth of about 250 ft. the rock is fairly sound.[2] On the basis of these

1. J. V. Davies, "The Astoria Tunnel under the East River for Gas Distribution in New York City," Paper No. 1359, Trans. Am. Soc. C. E., Vol. 80, (1916), pp. 594-674.
2. Berkey, Ch. T., "Geology of the New York City (Catskill) Aqueduct." Education Dept. Bulletin No. 489, Albany, New York, February 15, 1911.

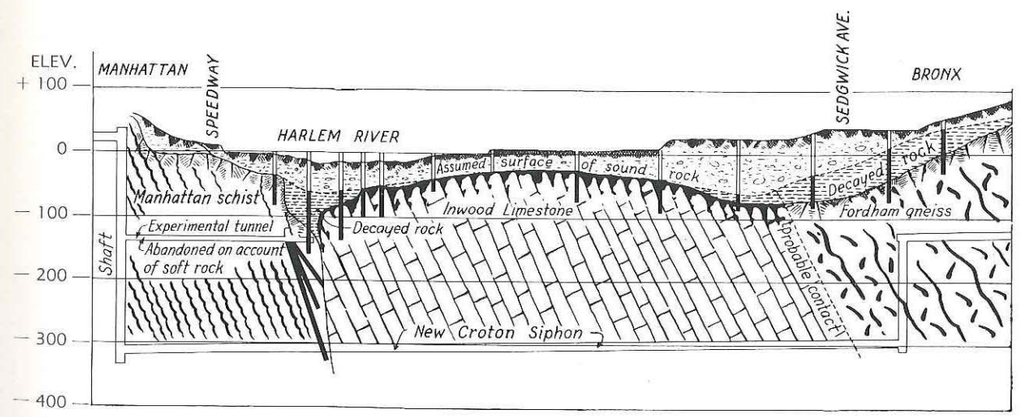


Fig. 12—Improvement or rock condition along cross fault with increasing depth.

Borings and an exploratory drift disclosed difficult rock conditions through a cross fault at an elevation of —120 ft. It was decided to drive the tunnel at —300 ft. where no great difficulty was encountered.

NEW CROTON AQUEDUCT, HARLEM RIVER SIPHON
Board of Water Supply, New York City — *New York State Museum*

findings it was decided to drive the tunnel at a depth of about 300 ft. below the level of the Harlem River, and no difficulties were encountered. If the geological survey and the subsequent borings had not been made, the tunnel would have been driven at a higher level and, as a consequence, the difficulties associated with traversing the cross fault might have been very serious.

Hudson River crossing of Catskill Aqueduct

Investigations similar to those on the Harlem River were made prior to choosing the site for the siphon which carries the water of the Catskill Aqueduct from the west to the east side of the Hudson River. Several sites were thoroughly explored before it was decided to cross the river at Storm King.

CHAPTER 3

CHEMICAL DEFECTS OF ROCKS

ORIGIN AND OCCURRENCE OF CHEMICAL DEFECTS

Causes of chemical defects

The term *chemical defects* refers to those defects of a rock which are due to alteration or removal of rock constituents resulting from chemical reaction between these constituents and water. Many rock constituents, such as feldspar, are gradually decomposed even by chemically pure water. However, the water which enters the rocks is never chemically pure. It contains various gases and solids in solution which may considerably intensify its chemical effects. Water may enter the rock from the surface, or it may rise into the rock from great depth. The main supply channels are the joints and faults and from these the water enters the interstices between the mineral grains.

The volume porosity of the densest rocks, including igneous rocks, such as granite, commonly ranges between 0.5 and 2.0 per cent. In spite of the low porosity of these rocks their permeability is relatively high. It may even be considerably greater than that of soft clay or a laboratory specimen of rich concrete. Since the voids occupy a very small part of the entire mass of rock a small amount of seepage is associated with a high rate of flow through the voids. The chemical action of the percolating water is further enhanced by the fact, inferred from experimental data, that almost the entire surface of every mineral grain is in contact with the percolating liquid.

The chemical action of rainwater which enters the rock from the surface is commonly referred to *chemical weathering*, whereas warm water coming from depth is said to produce *hydrothermal alteration*.

Defects due to weathering

The action of rainwater which enters the rock is twofold. Chemically unstable rock constituents, such as feldspar, are decomposed and soluble constituents, such as calcium carbonate or the soluble products of chemical decomposition, are removed in solution. Both processes combined are called *chemical weathering* and the removal of the soluble constituents is known as *leaching*. On account of these processes the surface of the intact rock is almost everywhere covered with a mantle of leached or weathered rock. This mantle can be compared to the layer of rust which develops on the surface of a piece of steel. At the very surface of the earth the weathered material turns into top soil which represents the ultimate stage of rock weathering.

In warm, moist atmosphere every perishable substance, including rock, decays much more rapidly than in a cool, dry climate. Therefore, the rate of rock weathering increases in general from the poles towards the equator, and at every latitude it increases with increasing annual rainfall. In regions of moderate rainfall in the southeastern United States weathering extends locally to a depth of 100 ft. Exceptionally, even greater depths of weathering have been encountered.

On the other hand, in semi-arid parts of the western United States, with a similar topography, the depth of the weathered zone is very small.

Finally, in regions which were formerly covered with ice, such as North America north of the parallel through New York, there are large areas where the zone of chemical rock weathering is completely absent. The plow-action of the moving ice sheet removed in many places everything located above the boundary of the chemically

and mechanically intact rock. The time which elapsed since the retreat of the ice was too short to permit the formation of a new zone of chemical rock weathering.

Defects of hydrothermal origin

In some regions the voids of the rocks have been invaded by hot aqueous solutions coming from relatively great depth. Changes produced by these agents are known as *hydrothermal alteration*. The solution may permeate the entire mass of rock or it may merely invade a fault zone and the adjoining rocks. The chemical alterations produced by the solution depend on the chemical composition of the solution and the rock; and also on the depth and temperature at which the changes take place.

In some parts of the earth's crust, chiefly at great depth, hydrothermal alteration was constructive. Mechanical defects, such as open fissures, were healed by deposition of quartz. Soft rocks, such as shale or weak sandstone, were transformed into hard rocks, such as schist or gneiss. These constructive processes have already been mentioned in the preceding chapter. On the other hand, in some regions the hot solutions coming from below have completely deprived the rock of its innate strength and have produced far more radical and detrimental changes than those associated with weathering. In connection with tunneling, only the detrimental effects of hydrothermal alteration need to be considered.

Chemical defects in fault zones

Fault zones in which crushing took place constitute underground drains. Since they may be invaded by surface waters or by hot solutions coming from below, or by both, chemical alteration in such zones is likely to be more intense and more varied than in the zone of weathering or in the zone of hydrothermal alteration. It may take place at any depth.

Conditions for the development of zones of chemical alteration

Chemical alteration is most active where water percolates through the voids of the rock either intermittently or continuously. This condition determines the boundaries of the zones of chemical alteration. The water which produces the alteration comes either from the surface or from the interior of the earth.

The flow of water coming from the surface requires a hydrostatic head with reference to sea level. Therefore the chemical action of surface waters cannot extend beyond a moderate depth below this level. However, if a zone of rock weathering subsided and was buried below younger sediments, it may be encountered at any elevation. In some regions the younger sediments were transformed into rock and the older rocks together with the superimposed younger ones were again lifted and folded. In such regions a tunnel may pass from intact older rocks through a zone of chemical rock weathering into intact younger rocks.

In the zone of hydrothermal alteration the flow of water into the rock is maintained by a source of excess hydrostatic pressure located at great depth. Hence effects of hydrothermal alteration may be encountered at any depth below the surface.

SOLUTION PHENOMENA IN SOLUBLE ROCKS

Origin of underground drainage channels in limestone

The most commonly encountered soluble rock is limestone. Limestone remains practically unaltered between waterbearing joints, but year after year a certain

quantity of rock is carried away in solution by the water which circulates in the joint system. For purely mechanical and hydraulic reasons the process tends to create in the rock more or less well-defined zones of drainage. It finally produces a system of continuous caves comparable to the brook and river channels which constitute the surface drainage system above insoluble rocks.

Absence of surface runoff in a limestone region is a sure indication of the existence of a natural system of drains, because the water must go somewhere. The character of the drainage system depends primarily on its degree of maturity. In its youth it consists of a fairly uniform system of water-bearing joints. Ultimately it becomes a fairly well defined and continuous system of caves separated from each other by almost impermeable rock. The surface waters enter the drainage system through joints or natural wells. The geologist can usually predict the general character and the discharge capacity of the system, but the location of the drains, including the main channels, can hardly be ascertained by surface observation in advance of construction.

It is doubtful whether extensive solution channels can be leached out below the lowest water table. But on account of a subsequent rise of the water table, extensive caves may be and have been encountered at a considerable depth below the lowest water table, and even below sea level. Caves may be partially or even completely filled by the insoluble constituents of the dissolved limestone. The fill usually consists of a reddish clay.

Hence, in limestone containing an underground drainage system the tunnel builder may encounter open caves or accumulations of limestone blocks or fragments. The interstices between the fragments may be open or filled with reddish clay. The difficulties due to the sudden inrush of large quantities of water or a mixture of water and sand or clay may be exasperating. But between the zones of drainage the rock is sound and even in broken zones pressure is never a serious problem.

Disintegration of dolomitic limestones

Dolomitic limestones consist of an aggregate of calcite and dolomite crystals. Both minerals are soluble in water, but the solubility of calcite is very much higher than that of dolomite. In some regions the calcite contained in such limestone has been removed by solution, whereas the dolomite crystals are still present. Such a process transforms the rock into an aggregate of dolomite crystals, with little cohesion. When the rock is blasted, it disintegrates into a sharp sand, similar to the crushed quartz which covers the floor of the tunnel in *Fig. 8.*

DECOMPOSITION OF CLASTIC ROCKS

Clastic rocks are accumulations of rock fragments which have subsequently been turned into rock by cementation or local intergrowth. If the rock fragments as such are not subject to perceptible chemical alteration, the process of rock decomposition merely resolves the rock into its original constituents. This statement applies to quartz conglomerates, quartz, sandstone and shale. Since shales disintegrate into materials similar to the silt or clay from which they were derived, the properties of a decomposed shale may range between those of rock flour and those of clay.

In contrast to quartz, sandstone and shale, some sandstone contains a high percentage of unstable minerals such as feldspar. Such rocks are altered to clay or to a clay-like material.

DECOMPOSITION OF METAMORPHIC ROCKS

Products of weathering of metamorphic rocks

In connection with the effect of weathering on metamorphic rocks, distinction must be made between rocks containing chiefly quartz or calcite such as quartzite and marble, and rocks consisting chiefly of micaceous minerals, such as chlorite and mica schist. Quartzite is unaffected by weathering and marble is subject chiefly to solution. On the other hand, most schists turn into a material which can hardly be distinguished from a clay. In some schists the transformation seems to be almost entirely due to a loss of cohesion and the mechanical breaking up of the original constituents of the rock, whereas in others part of the clay was produced by chemical changes, but the final product of the alteration invariably has the properties of clay.

Squeezing and swelling rock

The mechanical properties of a metamorphic rock altered into a clay-like substance can be as widely different as those of a sedimentary clay. Every soft clay and many stiff clays tend to squeeze into the tunnel wherever they are not supported. Therefore decomposed metamorphic rocks with clay characteristics are commonly referred to as *squeezing rock* or *squeezing ground*. However, the manifestations and the causes of the squeeze can be very different for different clays and decomposed rocks.

In most clays and most decomposed rocks the squeeze is merely due to a slow, viscuous flow of the material toward the tunnel, at almost constant water content. Less commonly the squeeze is chiefly due to an increase of the water content of the material, associated with expansion. If the squeeze of a decomposed rock is chiefly due to expansion, the rock is referred to as a *swelling rock*. Swelling clays and swelling rocks are likely to exert much heavier pressure on tunnel supports than clays and rocks without any marked swelling tendency.

The swelling capacity of both clays and decomposed rocks depends on several factors, including the type of clay minerals which they contain. The most conspicuous swelling tendencies are encountered in clays and decomposed rocks containing a high percentage of clay minerals of the montmorillonite group. Clays which consist chiefly of clay minerals of this group are known as *bentonites*.

DECOMPOSITION OF IGNEOUS ROCKS

Cause and effects of chemical alteration of igneous rocks

In contrast to schists, igneous rocks in an unweathered state do not contain a high percentage of platy minerals. Hence in a crushed but chemically unaltered state, these rocks have the property of an ordinary sand. Yet every igneous rock contains a high percentage of chemically unstable minerals, the feldspars. Chemical alteration commonly changes these into clay minerals. Hence the final product of the alteration of igneous rocks like that of a schist generally has the character of a clay. If encountered in a saturated state, it may exhibit the properties either of a squeezing or a swelling rock. The specific properties of the altered product depend on several factors including the type of clay minerals which originated during the process of alteration. Experience indicates that the swelling type of altered granites is most frequently encountered in zones which have been subject to hydrothermal alterations.

Serpentinisation

The internal decay of some dark colored igneous rocks, such as diabase, is associated with the formation of a greenish mineral known as *serpentine*. Experience

indicates that the occurrence of large quantities of serpentine in a rock may be associated with exceptionally heavy pressure on the tunnel support. This association probably results from the fact that the alteration to serpentine involves a volume expansion which creates severe internal stresses in the body of rock subject to alteration.

SYMPTOMS OF CHEMICAL ROCK ALTERATIONS

Characteristics of hand specimens of altered rock

The first symptoms of chemical decay of a rock are lack of lustre, local staining and dull sound when struck with a hammer. A fragment of decomposed rock, freshly recovered, may appear to be fairly or even entirely sound. However, if the fragment is exposed to the atmosphere for several weeks or months, it gradually disintegrates. This process is commonly referred to as *weathering or disintegration*.

Slaking at tunnel walls

The particles of a chemically altered rock in a disintegrated state are of different size. Some of them are crumbly and the heap of rock fragments is likely to contain some clay. A chemically intact rock weakened by a network of closely spaced joints also disintegrates, but the fragments are sound and of approximately equal size. The symptoms of chemical rock alteration are so important that they should be carefully observed and described in detail in both the daily reports and in papers dealing with construction operations.

If a cylindrical specimen of a chemically altered rock is submitted to a simple compression test by loading it, it is likely to disintegrate under relatively low pressure, before its height has decreased perceptibly. This property of altered rock is chiefly responsible for the 'slaking' of such rocks at the walls of tunnels (see *Chapter 4.)*

CHEMICAL ALTERATIONS IN RELATION TO TUNNELING

Tunneling through zones of chemical alteration

Chemical defects of rocks are commonly associated with mechanical defects such as jointing or faulting. Chemical weathering starts at the walls of fissures and proceeds towards the interior of the blocks located between the cracks. Therefore it is not uncommon that an altered rock which can be excavated with pick and shovel is interspersed with large, usually rounded, boulders of fairly intact rock requiring blasting.

In order to visualize the general character of thoroughly weathered or decayed rock it should be remembered that the fine grained sediments such as silt and clay are nothing else but the final product of chemical rock weathering. In connection with clastic and many metamorphic rocks, chemical weathering involves a return to the original state. Hence those sections of a tunnel which are located within zones of intense chemical alteration are nothing but sections of earth tunnels located between sections of rock tunnels. In some regions the transition from rock to earth is abrupt and in others it is gradual, but in any event it requires a change of both the method of attack and the type of support.

Sequence of zones of chemical alteration

When a tunnel is driven through a ridge it must pass on both sides through the zone of rock weathering. Within this zone the rock may have lost part or even all of its strength. If the ridge is low, the entire tunnel may be located within this zone. Between the sections located in the zone of rock weathering, chemically altered rock may again be encountered wherever the tunnel intersects a fault zone or a zone of hydro-

thermal alteration. The tunnel may even pass through an old zone of chemical rock weathering located beneath chemically intact younger rocks.

Estimate of boundaries of altered zones

The length of the tunnel sections located within the zone of surface weathering cannot be predicted with any degree of accuracy from surface indications, because it depends not only on the type of rock and on the character and history of the local topography, but also to a large extent on the exposure and other purely accidental factors. The following observations illustrate this fact.

U. S. Bureau of Reclamation

COLORADO—BIG THOMPSON PROJECT, ADAMS TUNNEL THROUGH THE CONTINENTAL DIVIDE

West Portal, near Granby, Colo.

Tunnel dimensions: driven bore 11'-9" wide by 12'-1" high, horseshoe; finished diameter 9'-9".

Support: continuous ribs with invert struts, 6" H-beams, 22.5 lbs. spaced from 1 ft. to 5 ft. centers.

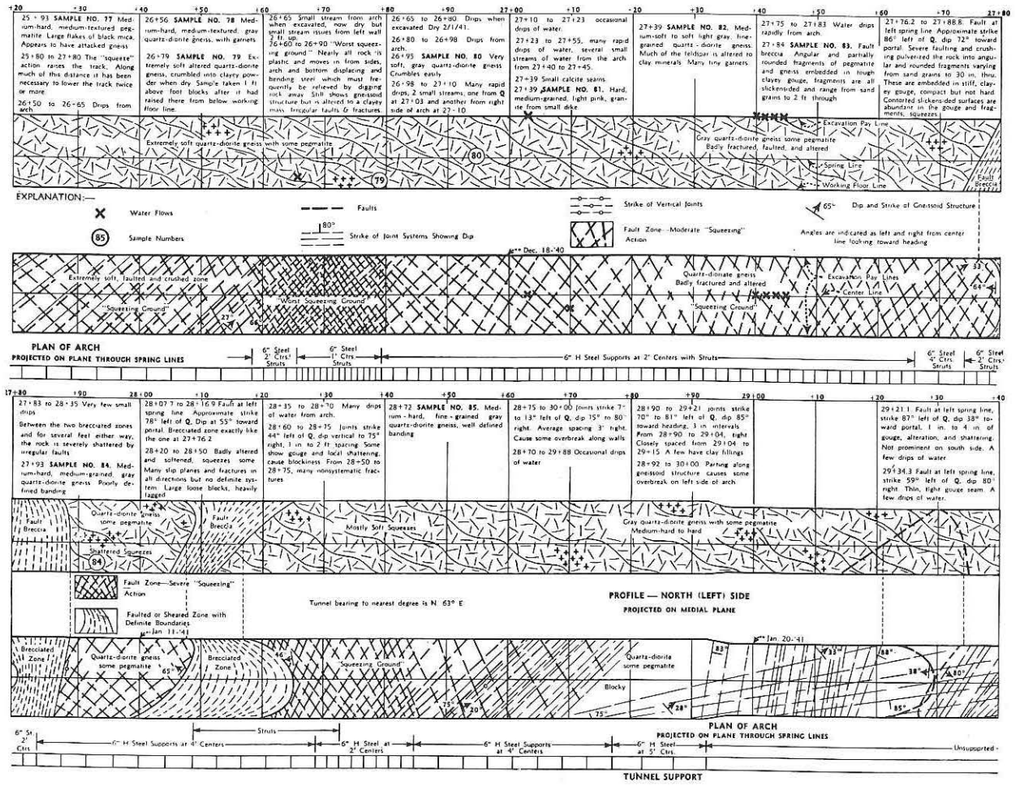

EXPLANATION:—
Water Flows
Sample Numbers
Faults
Strike of Joint Systems Showing Dip
Strike of Vertical Joints
Fault Zone—Moderate "Squeezing" Action
Dip and Strike of Gneissoid Structure
Angles are indicated as left and right from center line looking toward heading
Extremely soft quartz-diorite gneiss with some pegmatite
Extremely soft, faulted and crushed zone
"Squeezing Ground"
Minor Squeezing Ground
Quartz-diorite gneiss Badly fractured and altered
Excavation Pay Line
Spring Line
Working Floor Line
Center Line
Fault Breccia
PLAN OF ARCH
PROJECTED ON PLANE THROUGH SPRING LINES
6" Steel 2' Ctrs' Struts
6" Steel 1' Ctrs' Struts
6" H Steel Supports at 2' Centers with Struts
Fault Breccia
Quartz-diorite gneiss some pegmatite
Shattered Squeezes
Mostly Soft Squeezes
Gray quartz-diorite gneiss with some pegmatite Medium-hard to hard
Fault Zone—Severe "Squeezing" Action
Faulted or Sheared Zone with Definite Boundaries
Tunnel bearing to nearest degree is N. 63° E.
PROFILE — NORTH (LEFT) SIDE
PROJECTED ON MEDIAL PLANE
Brecciated Fault Zone
Brecciated Fault Zone
Quartz-diorite gneiss some pegmatite
Squeezing Ground
Quartz-diorite some pegmatite
Blocky
Struts
6" H Steel Supports at 4' Centers
6" H Steel at 2' Centers
6" H Steel Supports at 4' Centers
6" H Steel at 5' Ctrs
Unsupported
TUNNEL SUPPORT

Fig. 13—Erratic conditions of rock in fault zone.

The geologists' record of rock conditions in this tunnel illustrates the highly erratic sequence of rock conditions that may be encountered in a fault zone. The spacing of the steel ribs is an indicator of the conditions encountered. In the worst squeezing ground they were placed at 1 ft. centers and yet were distorted and had to be reinforced by knee braces as shown in Fig. 64. Yet only 230 ft. further on, no support was required.

In a valley in granite in central France the rock conditions have been explored by driving drifts into the sides of the valley. One side was very steep, whereas the other one rose at a gentle slope. On the steep side sound granite was encountered at a distance of about 4 ft. from the portal. On the gentle side excavation had to be made with pick and shovel through weathered granite and no sound material was met within a distance of 65 ft. from the portal. In another valley in granite it was found that the granite was completely decomposed to a depth of about 180 ft. measured at right angles to the slopes. The decomposed granite contained fragments of fairly sound rock with a diameter up to one foot.

Forecasts of rock conditions in fault zones involve similar uncertainties. The location and orientation of faults or fault zones is commonly disclosed by the results of the geological survey, but the type of chemical alterations and the width of the altered zone associated with the faults can hardly be predicted. In some instances it was found that the width of the crushed and altered zone decreases rapidly with increasing depth below the surface as shown in *Fig. 12*. In other locations no such tendency exists. Fault zones in igneous and metamorphic rocks may be associated with a rather erratic sequence of fairly sound and of completely decomposed squeezing or swelling rock. This condition is illustrated by *Fig. 13*. This figure represents part of the geologist's record of the rock conditions encountered in a fault zone during the construction of the Continental Divide Tunnel on the Big Thompson Project in Colorado, U. S. Bureau of Reclamation.

Least predictable are the rock conditions in tunnels through rocks such as granite which are subject to hydrothermal alteration. Both portals of such a tunnel may be located in perfectly sound rock. Nevertheless, on account of hydrothermal alterations in the interior of the mountain, undisclosed by surface evidence, the major part of the tunnel may require heavy support. Such surprises were experienced in various localities, including the Sierra Nevada in California.

Fig. 14—Vaulted overbreak caused by decay of roof support.

When originally constructed this tunnel was supported by timber sets, the posts and wall plates of which appear on the right. Work was stopped in 1885. In the intervening 54 years the timber roof rotted out and gradual dropping of shale from the roof created the vault shown in this photograph.

Pennsylvania Turnpike Commission
PENNSYLVANIA TURNPIKE
RAYS HILL TUNNEL
near Breezewood, Pennsylvania
Contractor: Mason & Hanger Company.

CHAPTER 4

LOAD ON TUNNEL SUPPORTS

INFLUENCE OF ROCK CONDITION ON ROCK LOAD

Rock load and earth pressure

Tunnel supports in rock tunnels are said to be acted upon by a rock load, whereas the tunnel supports in earth tunnels are acted upon by earth pressure. The term *rock load* indicates the height of the mass of rock which tends to drop out of the roof. If no support is supplied this mass of rock drops into the tunnel by increments whereby the roof assumes in the course of time the character of an irregular vault. Yet this vault may remain stable for a long time as demonstrated by the roof of natural caves. *Fig 14* shows such a vault. It was formed during a few decades by the dropping of rocks out of the roof of a tunnel after the roof support had rotted away On the other hand, the term *earth pressure* indicates the pressure exerted by a cohesionless or plastic material onto the tunnel support. If no support is installed, this material invades the tunnel either rapidly or slowly and the process continues until the tunnel has disappeared. The rock load depends on accidental details such as the spacing and the orientation of the joints, which may change from point to point, whereas the earth pressure depends on the average properties of the material surrounding the tunnel.

Earth tunnel conditions in rock tunnels

Wherever the rock is chemically intact and no more than moderately jointed, the roof of the tunnel is either self-supporting or else it requires no more than a tunnel support capable of sustaining moderate rock load. The sides of the tunnel are commonly stable. Exceptions to this rule will be mentioned. On the other hand, wherever the rock is completely crushed or decomposed, typical earth tunneling conditions will be encountered. This is not surprising, because "earth" is merely the final result of rock disintegration and chemical rock decomposition. Hence if a tunnel crosses a ridge or spur containing fault zones and zones of chemical alteration, it may consist of rock tunnel sections separated from each other by sections in which the methods of earth tunneling must be used.

Statistically by far the major part of the footage of existing rock tunnels has been driven under rock tunneling conditions. However, in some regions of geological disturbances the local occurrence of earth tunnel conditions is probable and in others it is certain. In some important tunnels, such as the Mono Craters Tunnel, a large part of the tunnel had to be constructed under earth tunneling conditions. Tunnel hazards and excess cost of construction are commonly associated with those tunnel sections in which earth tunneling conditions prevail. Hence the earth pressure on the earth tunnel sections requires at least as much attention as the rock load on typical rock tunnel sections.

Principal types of loading conditions

In perfectly or almost perfectly intact rock no support is required unless popping is encountered. In stratified or moderately jointed but otherwise intact rock the tunnel support serves its purpose if it is able to sustain a moderate rock load. In crushed rock the loading conditions are similar to those to be encountered when mining through sand; and in zones of rock decomposition they are similar to those in tunnels through clay. Tunneling through clay may be very easy or very difficult, depending on the

character and degree of compaction of the clay. An equally wide range of conditions is encountered when mining through decomposed rock. The behavior of the worst types of decomposed rock are indicated by the terms *squeezing* and *swelling* rock.

The transition from solid rock to earth-like rock may be abrupt or gradual. Rock intermediate between moderately jointed and crushed rock is commonly called *blocky* and *seamy rock*. No specific terms are used for rocks intermediate between intact and completely decomposed ones. The words *discolored* for the incipient and *crumbly* for the more advanced stages would be appropriate.

In the following articles only the most common types of loading conditions will be considered, starting with those in intact rock and ending with those in squeezing and swelling rock. The description of the loading conditions will be preceded by a discussion of the state of stress in rock prior to tunneling, because this state has a decisive influence on the reactions of the rocks on the tunneling operations.

STATE OF STRESS IN ROCK PRIOR TO TUNNELING

The load on tunnel supports depends more or less on the state of stress in the rock prior to tunnel construction. In this connection distinction must be made between vertical loads and horizontal pressures. The vertical load on a horizontal section through a mass of rock is equal to the weight of the rock located above this section. The horizontal unit pressure at the elevation of such a section may range between wide limits. In accordance with practice prevailing among tunnel men vertical loads will be expressed by the thickness **H** in feet of rock. The vertical load in pounds per sq. ft. is equal to **H** times the unit weight, **w**, in pounds per cu. ft. of the rock, or **Hw**. The unit weight of the rock can be determined by a sample test. It ranges between 165 and 180 lbs. per cubic foot. On the other hand the horizontal pressures p_h will be expressed in pounds per sq. ft.

The ratio between the horizontal pressure, p_h, and vertical load **Hw** in a mass of rock depends primarily on the geological history of the rock. In an undisturbed mass of rock, the horizontal unit pressure at any given point is likely to be considerably smaller than the vertical load at the same point. In a folded mass of rock, the horizontal pressure depends on whether the horizontal forces which produced the folding have already disappeared or whether they are still active. If they have disappeared, the horizontal pressures may be as low as in an undisturbed mass of rock. On the other hand, if the horizontal forces are still active, the horizontal pressure at any depth can be close to the compressive strength of the rock. Since we have no reliable means for determining the state of stress in the interior of a mass of rock, the existence of heavy horizontal pressure can only be inferred from its visible manifestations, such as popping rock at a moderate depth below the surface.

TUNNELS THROUGH INTACT ROCK

Stresses at tunnel walls

By theory it has been shown that the effect of excavating a tunnel through intact rock on the state of stress in the rock rapidly decreases with increasing distance from the tunnel walls. At the walls the normal stress at right angles to the wall (radial stress) is zero and the normal stress acting in a circumferential direction (circumferential stress) is roughly equal to twice the stress which acted at the same point prior to tunneling. With increasing distance from the tunnel walls the radial stress increases, the circumferential stress decreases and at a distance equal to about the diameter of the tunnel, the state of stress in the rock is practically unaltered.

At the walls the state of stress in the rock is similar to that in an unconfined rock specimen subject to axial compression in a testing machine. The rock does not fail in the tunnel until the circumferential stress becomes equal to the unconfined compressive strength of the rock. In rocks which are not acted upon by horizontal forces, such as those which lifted up the mountain chains, the circumferential stress does not exceed about twice the overburden pressure. On account of this condition and the great strength of intact rocks, failure by crushing would not occur in porous sandstone at a depth of less than about 5000 ft. The critical depth increases with the increasing strength of rock and reaches more than 35,000 ft. for granite and other igneous rocks.

Popping in tunnels through intact rock

The term *popping rock* refers to rock formations from which thin slabs of rock are suddenly detached after the rock has been exposed in a quarry or a tunnel. Popping normally occurs only in hard rocks in an intact state. In tunnels the slabs are popped off either from the sides or from the roof of the tunnel. Popping has been encountered only in hard and brittle rocks. It has invariably been found that the detached slabs do not fit the surface from which they popped. This fact indicates that the rock to which the slab was attached is in a state of intense elastic deformation.

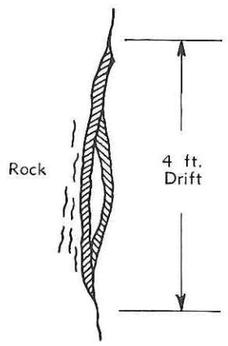


Fig. 15—Popping rock

Fig. 15 illustrates popping as observed in a drift during the construction of the Simplon Tunnel in Switzerland. The shaded areas indicates slabs on the verge of popping off, whereas the black lines represent the assumed position of the cracks which precede the detachment of succeeding slabs.

In some regions the elastic deformations disclosed by popping appear to be due to the fact that the horizontal pressure which led to the formation of geologically young mountain chains is still active. However, in other regions it may be due to still unknown causes.

Whatever the underlying physical causes may be, tunnels in popping rock require both temporary and permanent lining, to protect the workmen from flying rock fragments.

Protection against popping rock

Fig. 15 shows that the popping is preceded by an inward bending of the slabs. If a sufficient counter force is applied at right angles to the slabs to prevent bending, the slabs would remain in place and fail, at a higher load, by shear or by crushing. The pressure required to prevent the inward bending is very small. Therefore, any tunnel lining which is capable of sustaining an external load of about 400 lbs. per sq. ft. should suffice to prevent slab fragments from being thrown into the tunnel. Since the popping occurs most frequently at the sides of the tunnel, the footings of the ribs should be secured against horizontal displacement.

Commonly the process of popping involves only a breakage of the rock in the immediate vicinity of the tunnel walls. But if it initiates a progressive general breakage of the rock surrounding the tunnel, the tunnel support should be strong enough to carry the load of blocky and seamy rock (see below). In any event the lining should be tightly wedged against the walls of the tunnel, or the space between the lining and the rock should be back packed.

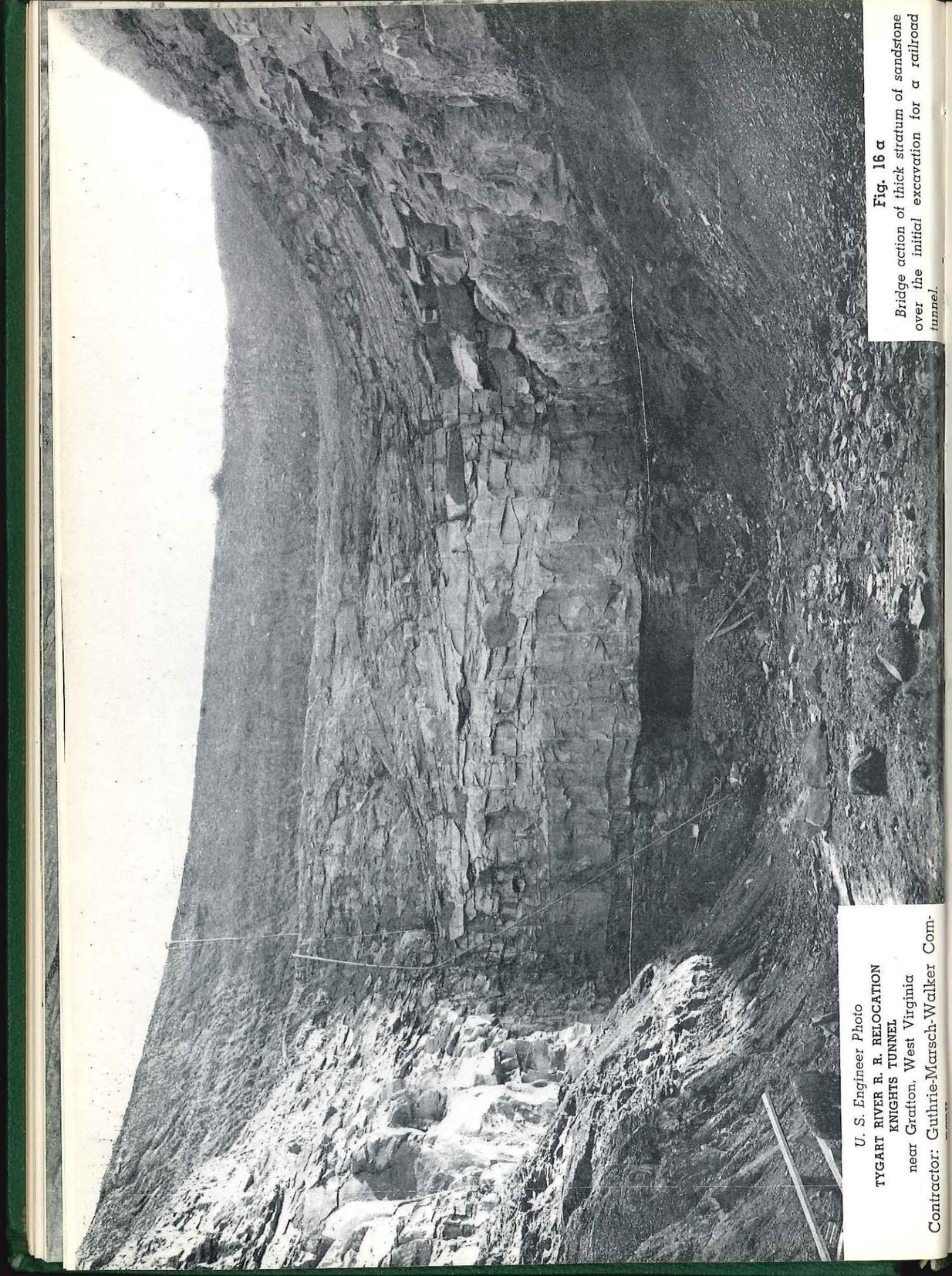


Fig. 16 a
Bridge action of thick stratum of sandstone over the initial excavation for a railroad tunnel.

U. S. Engineer Photo
TYGART RIVER R. R. RELOCATION
KNIGHTS TUNNEL
near Grafton, West Virginia
Contractor: Guthrie-Marsch-Walker Com-

TUNNELS IN UNWEATHERED STRATIFIED ROCKS AND IN SCHISTS

Sources of weakness

Almost every stratified rock breaks readily along bedding planes. Therefore the bedding planes constitute a source of mechanical weakness. In schists the cleavage planes have a similar effect. In addition to these innate mechanical defects, every stratified rock and every schist contains at least two sets of joints oriented at approximately right angles to the planes mentioned before. These will be referred to as *transverse joints.*

Bridge action in rocks with horizontal layers

If the spacing of the transverse joints is greater than the width of the tunnel, the rock layers bridge the tunnel like solid slabs as shown in *Fig. 16 a* and they are subject to bending under their own weight. If the bending stresses are smaller than the tensile strength of the rock, the roof is stable without any support, as shown in *Figs. 16 b* and *c*. *Fig. 16 c* also illustrates the benefits derived from an arch-shaped roof. The sides of the arch constitute corbels which reduce very considerably the free span of the roof slabs.

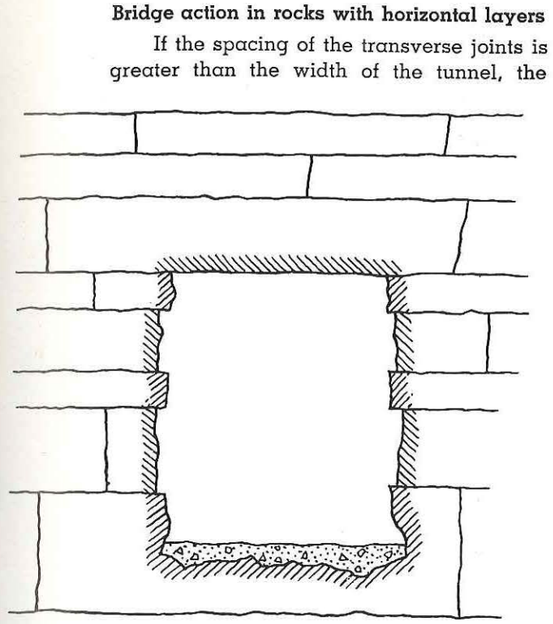


Fig. 16 b **Bridge action in rocks with widely spaced transverse joints.**

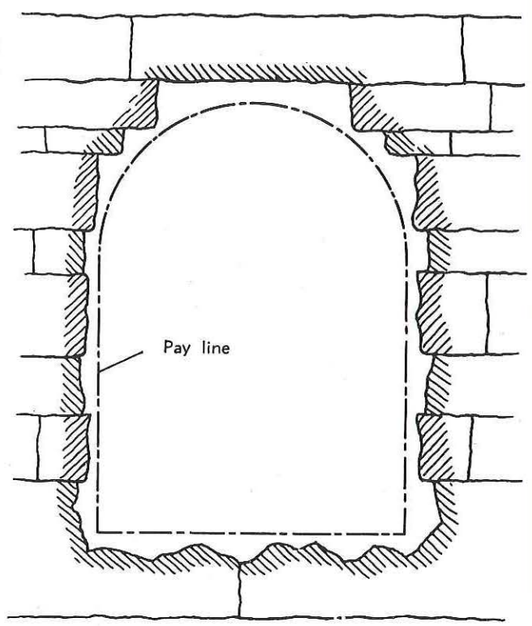


Fig. 16 c **Bridge action in rocks with closely spaced transverse joints.**

If the bending stresses in the rock layers above the tunnel exceed the strength of the rock, or if layers are weakened by transverse joints, they require support as shown in *Fig. 17.*

The corbel arch principle has been used in early days for reducing the free span of bridges as shown in *Fig. 18.* Heavy wooden beams, each one longer than the one below it, cantilever out from the abutments thereby reducing the free span of the bridge to a nominal figure.

Fig. 17—Tunnel in thin bedded, closely jointed sandstone.

On account of the close spacing of both joints and bedding planes of this "short breaking" rock, support was required to hold the roof. The temporary posting was later replaced by steel sets.

Pennsylvania Turnpike Commission
PENNSYLVANIA TURNPIKE
SIDELING HILL TUNNEL
West Portal, near Breezewood, Pennsylvania
Contractor: The Arundel Corporation.

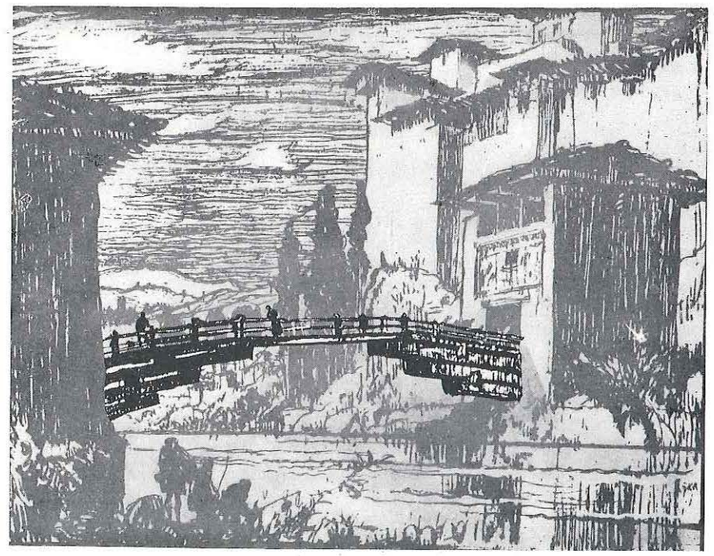


Fig. 18

After Frank Brangwyn, A. R. A.

Heavy cantilever beams used to reduce the free span of a wooden bridge. They are similar to the corbels at the haunches in the tunnel shown in Fig. 16 c.

Overbreak and rock load in horizontally stratified rock

The overbreak depends on several factors. Foremost among them are the following:

Spacing between the joints.

Shattering effect of blasting on the rock located beyond the payline.

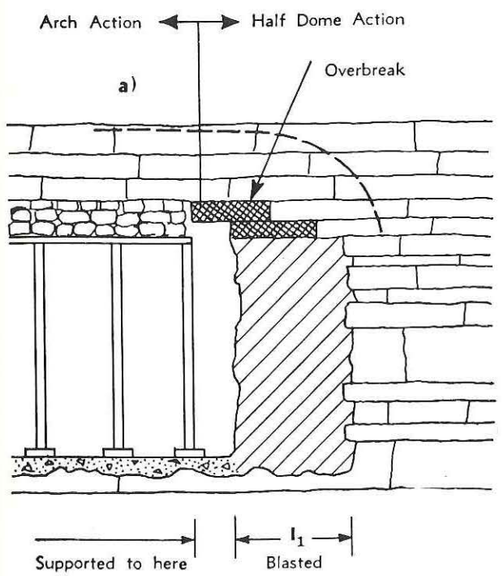
Distance between the working face and the roof support, and

Length of time which elapses between the removal of the natural support of the roof and the installation of the artificial support.

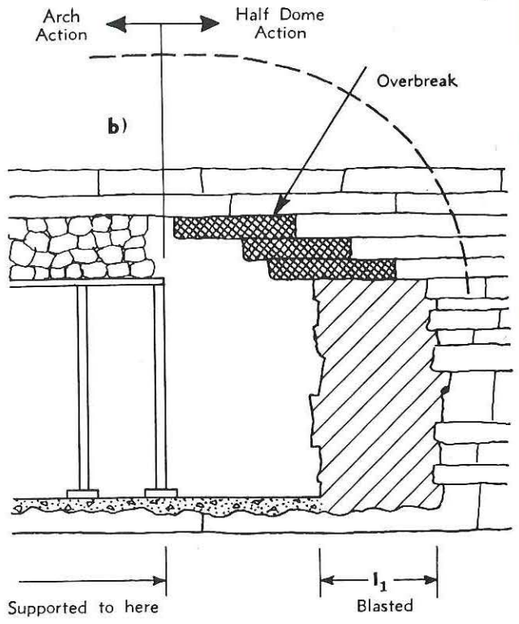
Figs. 19 a and *b* illustrate the influence of the distance between the working face and the supported roof on the overbreak in closely jointed, horizontally stratified rock. The smaller this distance, the smaller is the quantity of rock which is likely to drop out of the roof when the round is fired.

If *Figs. 19 a* and *b* are considered in conjunction with the transverse section of the tunnel illustrated in *Fig. 16 c*, it will be seen that the rock over the face constitutes *a half dome*. Within the limits of the joint spacing, the roof is supported by the rock ahead of the face and on the sides. Succeeding rounds remove the forward support from the half dome so that the rock above the roof is carried by bridge action or arch action.

If no roof support is constructed, a certain quantity of rock will drop from the roof whereupon the roof assumes the character of that of a natural cave. If the rock is

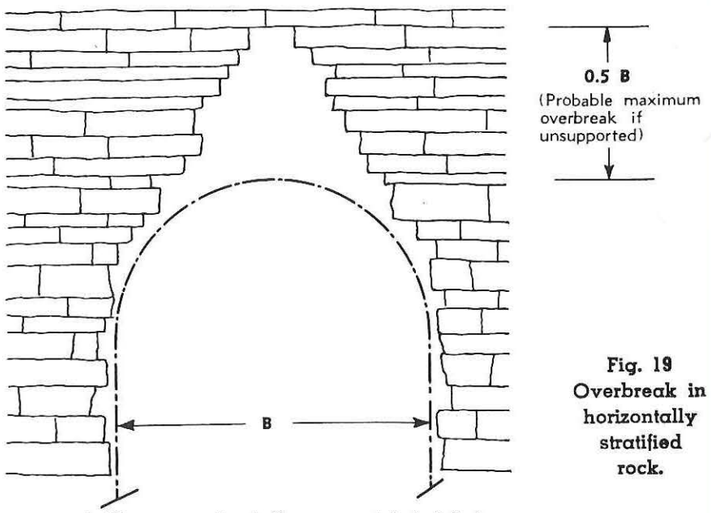


a) Overbreak if unsupported section is very short.



b) Overbreak if unsupported section is long.

composed of fairly thick strata with few joints, the roof will be flat as shown in *Fig. 14*. On the other hand if the strata are thin and weakened by many joints a peaked roof will be formed as indicated in *Fig. 19 c*. Yet the breakage will rarely if ever continue after the vertical distance between the top of a semi-circular payline and the top of the overbreak becomes equal to 0.5 **B** *as* indicated in *Fig. 19 c*. This condition determines the maximum value which the load on the roof support can assume.



c) Ultimate overbreak if no support is installed.

Fig. 19 Overbreak in horizontally stratified rock.

If the tunnel support is constructed and wedged soon after blasting, the friction forces on the sides of the rock fragments occupying the space between roof support and vault transfer part of the weight of this rock onto the rock located beyond the sides of the vault. Hence even the ultimate load, measured in feet of rock, on an

adequately constructed and backpacked roof support in closely jointed, horizontally stratified rock is likely to be much smaller than the value 0.5 **B**. On the other hand, if large empty spaces are left between the roof support and the roof, blocks will drop out of the roof, one by one, whereby the load on the roof support may increase to its maximum of 0.5 **B**.

In some rocks the spacing of the joints changes considerably from place to place. In such rocks the load on a well-wedged tunnel support may vary between zero and a maximum somewhat smaller than 0.5 **B**.

Overbreak and rock load in tunnels through vertical strata

In folded masses of rock the dip of the strata may range anywhere between 0° and 90°, and the strike may intersect the center line of the tunnel at any angle between 0° and 90°. In the following discussions it is assumed that the center line of the tunnel is parallel to the strike and that the strata are vertical as shown in *Fig. 20 a*. In rock with such a structure, the individual strata bridge the space between the heading

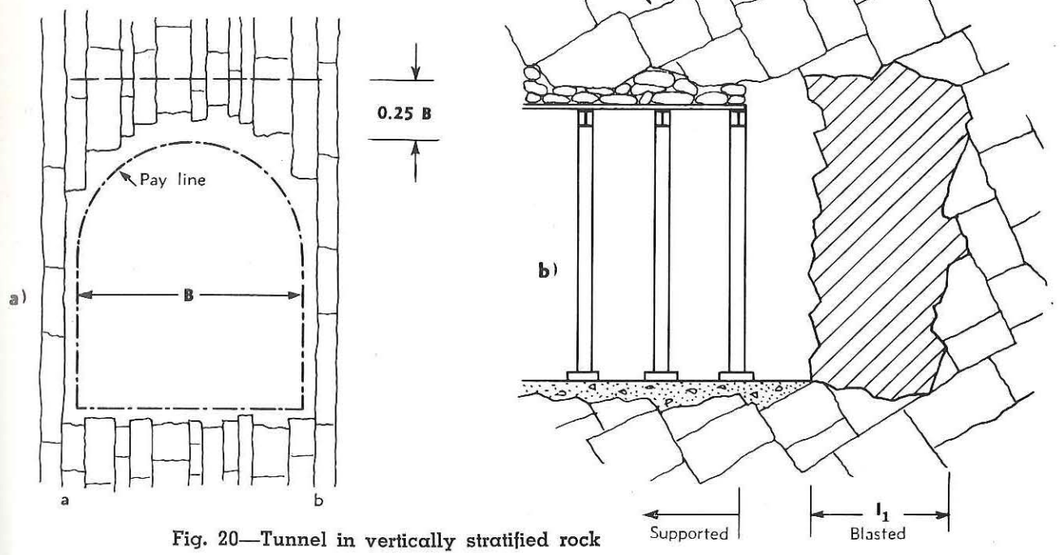


Fig. 20—Tunnel in vertically stratified rock

and the supported part of the tunnel, *Fig. 20 b*. Hence, if the joints are closely spaced, the amount of overbreak depends to a large extent on the distance between the working face and the supported roof.

The mass of rock located above the roof is held merely by the friction along the two bedding planes passing through **a** and **b** in *Fig. 20 a*. The roof has to carry the entire difference between this weight and the total friction forces. If these planes were perfectly even, the load on the roof support could be very important. However, in nature these surfaces are always more or less uneven. Hence the load on the roof rarely exceeds the weight of the rock which has been shattered and loosened by blasting.

There is no evidence that the upper boundary of the loosened rock is located at an elevation of more than about 0.25 **B** above the crown of the roof support. If no large empty spaces are left between the rock and the roof support, no subsequent loosening-up of the rock located above the roof can take place. On this assumption it seems safe to assume that the load on the crown of the roof support will not exceed 0.25 **B** ft. of rock.

Overbreak and rock load in tunnels through inclined strata

Fig. 21 is a section through a tunnel located in the inclined part of a fold whose axis is parallel to the center line of the tunnel. On account of the stratification the overbreak tends to produce a peaked roof as shown in *Figs. 21* and *22*. In *Fig. 22*,

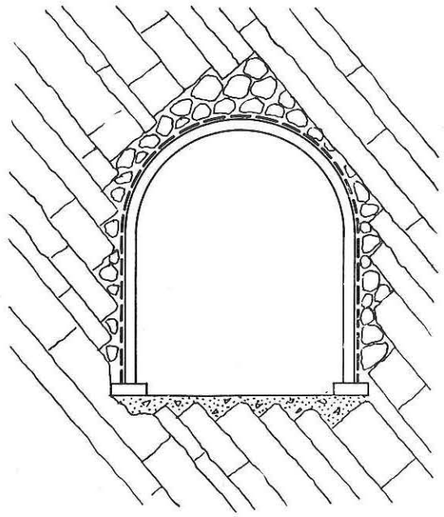
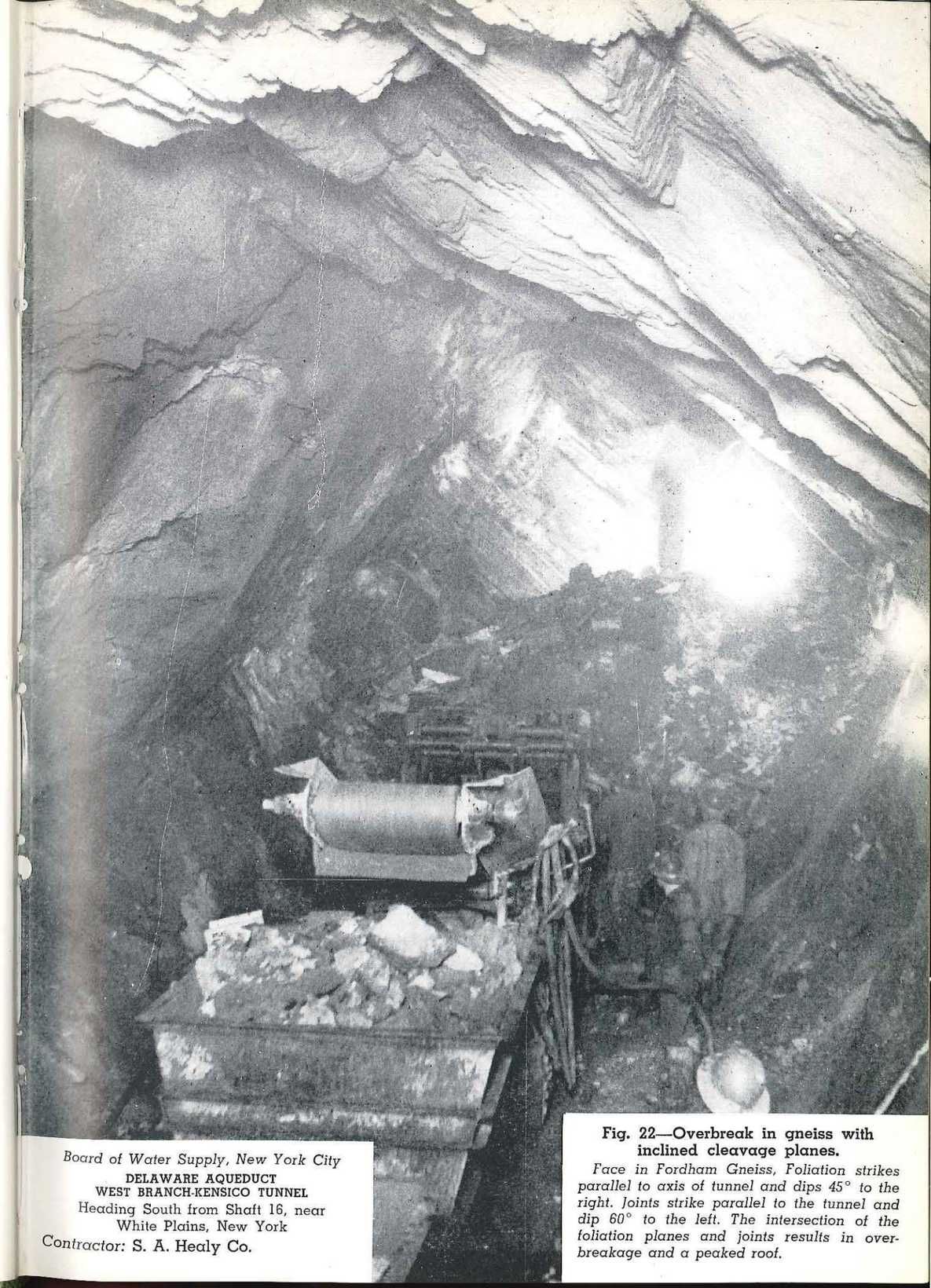


Fig. 21—Tunnel in rock whose strata are steeply inclined

illustrating the overbreak in a tunnel through gneiss, the right-hand side of the roof coincides with a cleavage plane or plane of foliation and the left-hand side coincides with joints.

If the bedding or cleavage planes rise at a steep angle to the horizontal, a wedge-shaped body of rock, **a e d** in *Fig. 23*, tends to slide into the tunnel and subjects the post at **a c** to bending. The lateral force, **P** per unit of length of the tunnel, which acts on the post can be estimated as indicated in *Fig. 23*. The estimate is based on the assumption that the rock indicated by the shaded area to the right of **c e** in *Fig. 23 a* has dropped out of the roof and that there is no adhesion between rock and rock along **d e**. On these assumptions the wedge-shaped body of rock **a d e** is acted upon by its weight, **W**, and the reaction **Q** on the surface of sliding **a d**. In order to prevent a downward movement of the wedge, the vertical post **a c** must be able to resist a horizontal force **P**. The reaction **Q** acts at an angle ø to the normal on the surface of sliding **a d**. The angle ø is the angle of friction between the wedge and its base. The weight **W** is known. The intensity of the forces **Q** and **P** can be determined by means of the polygon of forces shown in *Fig. 23 b*.



Board of Water Supply, New York City
DELAWARE AQUEDUCT
WEST BRANCH-KENSICO TUNNEL
Heading South from Shaft 16, near White Plains, New York
Contractor: S. A. Healy Co.

Fig. 22—Overbreak in gneiss with inclined cleavage planes.
Face in Fordham Gneiss, Foliation strikes parallel to axis of tunnel and dips 45° to the right. Joints strike parallel to the tunnel and dip 60° to the left. The intersection of the foliation planes and joints results in overbreakage and a peaked roof.

The angle of friction ø depends not only on the nature of the surfaces of contact at a d but also on the hydrostatic pressure in the water which percolates into the space between the two surfaces. Experience with slides in open cuts in stratified rocks indicates that the value of ø for stratified rocks with clay or shale partings may be as low as 15°. If no such partings are present, 25° seems to be a safe value.

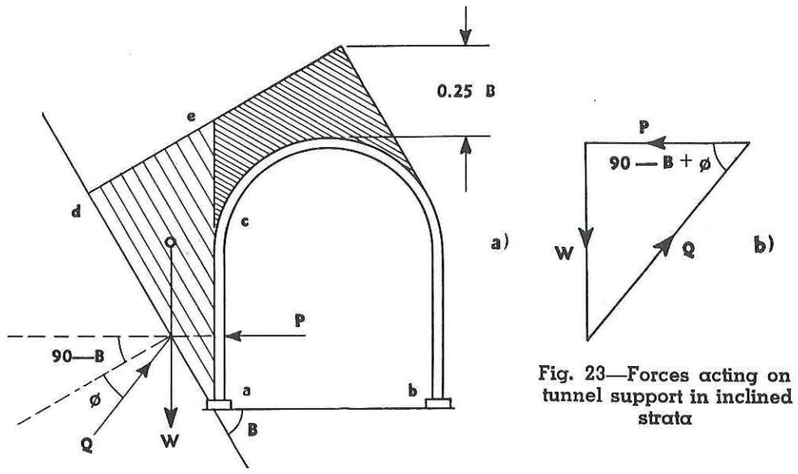


Fig. 23—Forces acting on tunnel support in inclined strata

The highest value for the unit pressure on the roof depends on the slope of the strata. For steep strata it will hardly exceed 0.25 B, whereas for gently inclined strata it may approach the value 0.5 B.

TUNNELS THROUGH MODERATELY JOINTED, MASSIVE ROCKS

Overbreak and rock load

One set of joints in massive rocks is commonly parallel to the ground surface. Overbreak and rock load conditions in such rocks are similar to those in stratified rocks. If the joints are oriented at random, absence of a roof support would ultimately lead to the formation of a vault-shaped roof as shown in *Fig. 24 a*. Whatever the orientation of the joints, the blocks located between joints below the zone of rock weathering are so intimately interlocked that they have very little freedom of movement. This is demonstrated by the fact that the vertical sides of tunnels through such rocks rarely require any lateral support. The roof is the only place in the tunnel where blocks are likely to become detached.

Owing to gradual adjustments in the state of stress in the rock adjoining the tunnel the interlocks between blocks may fail long after an unsupported tunnel is finished, but it is hardly conceivable that such failures would occur if the rock is tightly wedged against a tunnel support. Hence the greatest value which the rock load on such a support can assume will be considerably smaller than the weight of the body of rock located between the tunnel support and the upper boundary of the potential overbreak indicated in *Fig. 24* a which is 0.25 B. Hence the actual load on the crown may vary between zero and 0.25 B depending on the orientation and spacing of the joints. There is no reason for assuming that the load will increase with time, provided there are no large empty spaces between support and rock.

Popping

If the rock is in a state of intense elastic deformation, due to tectonic stresses or other causes, the connections or interlocks between blocks such as A and B in *Fig. 24 b* and their neighbors, may suddenly snap, whereupon the block is violently thrown into the tunnel. If such an incident occurs, it is necessary to provide the tunnel with the support prescribed for popping, intact rock (see article on intact rock).

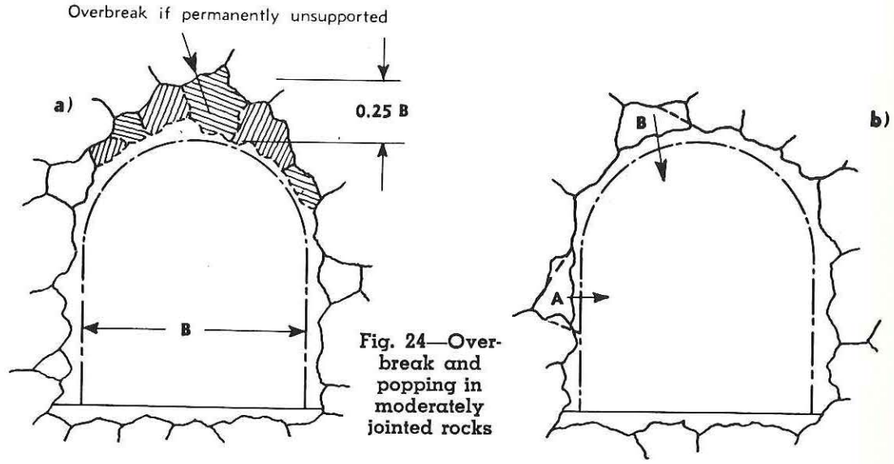


Fig. 24—Overbreak and popping in moderately jointed rocks

TUNNELS IN CRUSHED ROCK

General character of crushed rock

If a competent rock, such as quartzite or quartzitic sandstone, is subject to intense deformation, for instance by shear in a fault zone, it fractures to such an extent that it loses the capacity to form an unsupported vault bridging a cave or a tunnel. It may even be reduced to powder as if it had passed through a crushing machine.

As a tunnel approaches a zone of intense crushing, it passes through rock which is more and more intensely jointed and finally it enters a zone in which the rock resembles cohesionless sand. Yet, experience shows that even in sand and in completely crushed but chemically intact rock, the rock load on the roof support does not exceed a small fraction of the weight of the rock located above the roof and if the depth of the overburden is greater than about 1.5 times the combined width and height of the tunnel, the rock load is practically independent of depth. The cause of this phenomenon is commonly known as *arch action*.

Arch action in crushed rock

The term *arch action* indicates the capacity of the rock located above the roof of a tunnel to transfer the major part of the total weight of the overburden onto the rock located on both sides of the tunnel. The body of rock which transfers the load will briefly be referred to as the *ground arch*.

In order to investigate the cause of arch action and the laws which determine the rock load on the roof support, numerous model tests have been made with perfectly

cohesionless sand. A detailed description of these tests will be presented in a companion volume on *"Earth Tunneling with Steel Support."* The test results led to the following conclusions regarding the prerequisites for arch action and the factors which determine the load on the roof support in tunnels through crushed rock and cohesionless sand located above the water table.

(a) The arch action is the inevitable consequence of the local stress relaxation produced by mining operations. The mechanics of the arch action are illustrated by *Fig. 25.* In this figure the ground arch is represented by the shaded area **a c d b.** The ground arch has a width B_1. While the tunnel is being excavated and the support installed, the mass of crushed rock or cohesionless sand constituting the ground arch

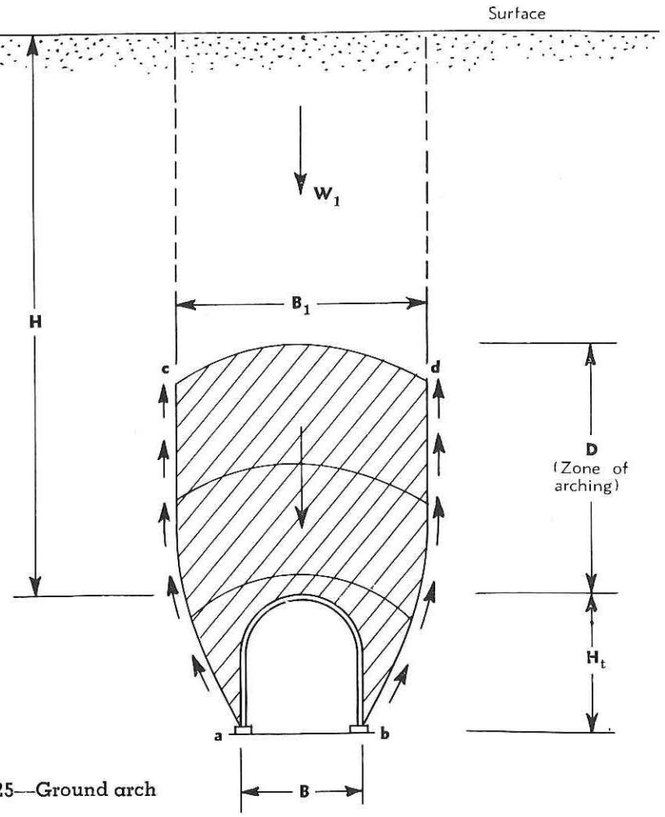


Fig. 25—Ground arch

tends to move into the tunnel. This movement is resisted by the friction along the lateral boundaries **a c** and **b d** of this mass. The friction forces transfer the major part of the weight of the overburden, with height **H** onto the material located on both sides of the tunnel and the roof support carries only the balance, equivalent to a height H_p.

(b) The thickness **D** of the ground arch is roughly equal to 1.5 B_1. Above the ground arch the pressure conditions in the rock remain practically unaffected by the tunnel operations.

(c) A very small downward movement of the crown of the tunnel suffices to reduce the rock load on the support of the intrados of the arch to a value $H_{p\ min}$, which is very much smaller than the thickness **D** of the ground arch. If the crown of the ground arch is allowed to subside still more, the rock load on the roof support increases and approaches a value $H_{p\ max}$ which, however, is also much smaller than **D.**

(d) After the roof support is installed and tightly backpacked, the rock load increases at a decreasing rate by about fifteen percent from H_p to $H_{p\ ult}$,

$$H_{p\ ult} = 1.15\,H_p \qquad (1)$$

(e) As the depth of the overburden on a tunnel with a given cross-section increases from zero, the roof load increases as indicated by curve **C** in *Fig. 26.* With increasing depth **H** the rock load approaches a value H_p which is independent of depth.

(f) The value H_p in *Fig. 26* increases approximately in direct proportion to the width B_1 of the ground arch, *Fig. 25,* everything else being equal. This relation can be expressed by the equation

$$H_p = C \times B_1 \qquad (2)$$

wherein **C** is a constant. The value of **C** depends on the degree of compactness of the crushed rock or sand and on the distance **d** through which the crown of the ground arch subsided while the tunnel was mined and the roof support was being installed. Numerical values for **C** will be presented under the following subheading.

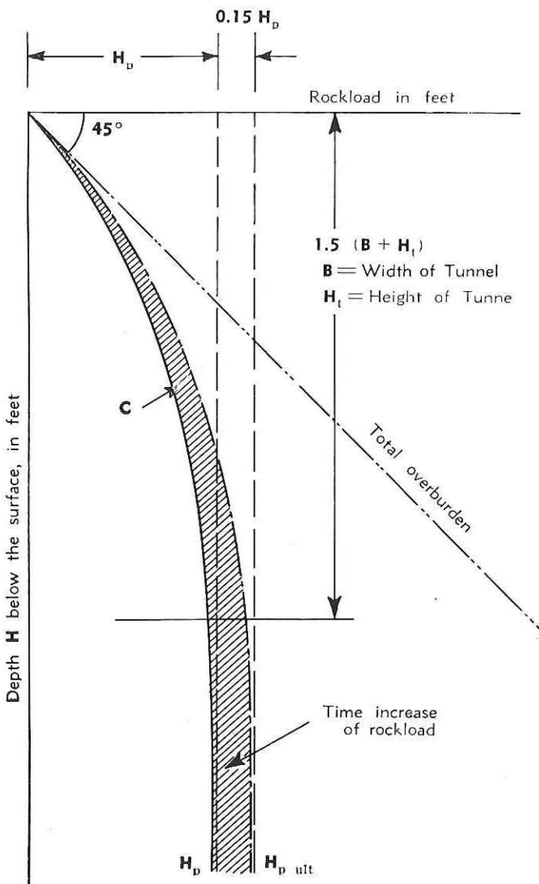


Fig. 26—Load-depth relationship on tunnel in sand

Rock load on roof support in tunnels through completely crushed rock or sand above the water table

Fig. 27 is a cross-section through a tunnel through chemically intact rock, crushed to sand, located at great depth below the surface. The lower boundaries of the mass of rock which tends to move into the tunnel (body **a b d c** in *Fig. 25*) rise from the outer edges of the bottom of the tunnel at a slope of about 2 (vertical) on 1 (horizontal). Hence the width B_1 of the ground arch, *Fig. 25,* is roughly equal to

$$B_1 = B + H_t \qquad (3)$$

wherein **B** is the width and H_t the height of the tunnel.

The rock load H_p is represented in *Fig. 27* by the rectangle **e f f₁ e₁**. The balance of the weight of the overburden is carried by the ground arch. The weight of the middle part **c d d₁ c₁** is transferred by the ribs of the tunnel support to the floor of the tunnel. The weight of the outer part acts as a surcharge on the top of the wedge-shaped bodies which tend to slide into the tunnel and increase the horizontal pressure exerted by these bodies.

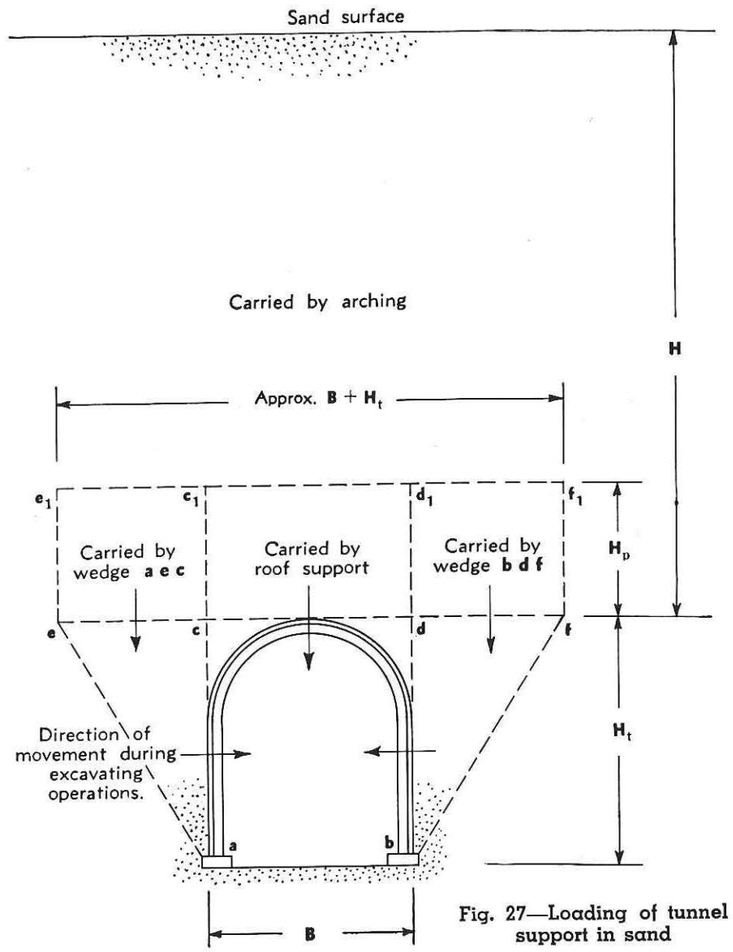


Fig. 27—Loading of tunnel support in sand

The rock load H_p is determined by eq. (2). According to the text accompanying this equation, the value of the constant **C** depends on the degree of compactness of the materials in which the tunnel is located and on the distance **d** through which the crown of the ground arch yielded before the support was installed. The distance **d** is not known and it can hardly be determined by practicable means. At a given width **B** of the tunnel it depends to a large extent on the skill of the miners and on the care with which the tunnel support is backpacked. The following numerical values are exclusively based on the results of the model tests with dry sand. Nevertheless it is

believed that they furnish fairly accurate information concerning the influence of the degree of compactness of the crushed rock and of the amount of yield associated with the mining operations on the intensity of the rock load.

Dense sand $H_{p\,min} = 0.27\ (B + H_t)$ for yield of $0.01\ (B + H_t)$ (4)

$H_{p\,max} = 0.60\ (B + H_t)$ for yield of $0.15\ (B + H_t)$ or more (5)

Loose sand $H_{p\,min} = 0.47\ (B + H_t)$ for yield of $0.02\ (B + H_t)$ (6)

$H_{p\,max} = 0.60\ (B + H_t)$ for yield of $0.15\ (B + H_t)$ or more (5)

The sand pressure on the sides of the tunnel support can be estimated by means of the earth pressure theory. In this way, it was found that the average unit pressure p_h on these sides is roughly equal to

$$p_h = 0.30\ w\ (0.5H_t + H_p) \tag{7}$$

in which **w** is the weight per cu. ft. of the sand.

After the tunnel support is installed and backpacked, both the rock load and the side pressure gradually increase by about 15 percent, regardless of the initial value of H_p.

Experience shows that the roof load in tunnels through crushed rock and sand above the water table is commonly much closer to the minimum than to the maximum values determined by the preceding equations. This fact indicates that the slight movement of the rock towards the tunnel, induced by the mining operations, fully satisfies the deformation condition for arching. Since a yield of the rock beyond the minimum required to produce arch action causes an increase of the load on the roof, the tunnel support should be as quickly and tightly backpacked as conditions permit.

Effect of seepage on arch action in sand and crushed rock

If a tunnel through sand or crushed rock is located below the water table the tunnel acts like a sub-surface drain and the water percolates through the voids or interstices of the surrounding material towards the tunnel. The effect of the percolating water on the arch action was investigated by means of model tests similar to those referred to under the preceding subheading. The sand located above the model of the tunnel roof was flooded. The tunnel roof was perforated and the water which percolated through the roof was continuously replaced. By measuring the pressure on the tunnel roof corresponding to different amounts of subsidence of the crown of the ground arch located above the roof it was found that the flow of water does not interfere with the arching action. But the load exerted by the percolating water roughly doubles the height H_p of the layer of sand whose weight exerts load on the roof.

Effect of seepage on bearing capacity of rib footings

If a tunnel through sand is located below the water table, the water percolates towards the tunnel as shown in *Fig. 28 a*. Part of the seepage percolates through the sand in an upward direction and enters the tunnel through the floor. In order to investigate the influence of such a flow on the stability of the floor and its capacity to sustain the load transferred through the posts onto the footings the experiment illustrated by *Fig. 28 b* was made. A layer of sand with thickness **H** was placed on a sieve located above the bottom of cylindrical vessel. Water entered the vessel from below at **a**, percolated through the sand in an upward direction and left the vessel at **b**. The

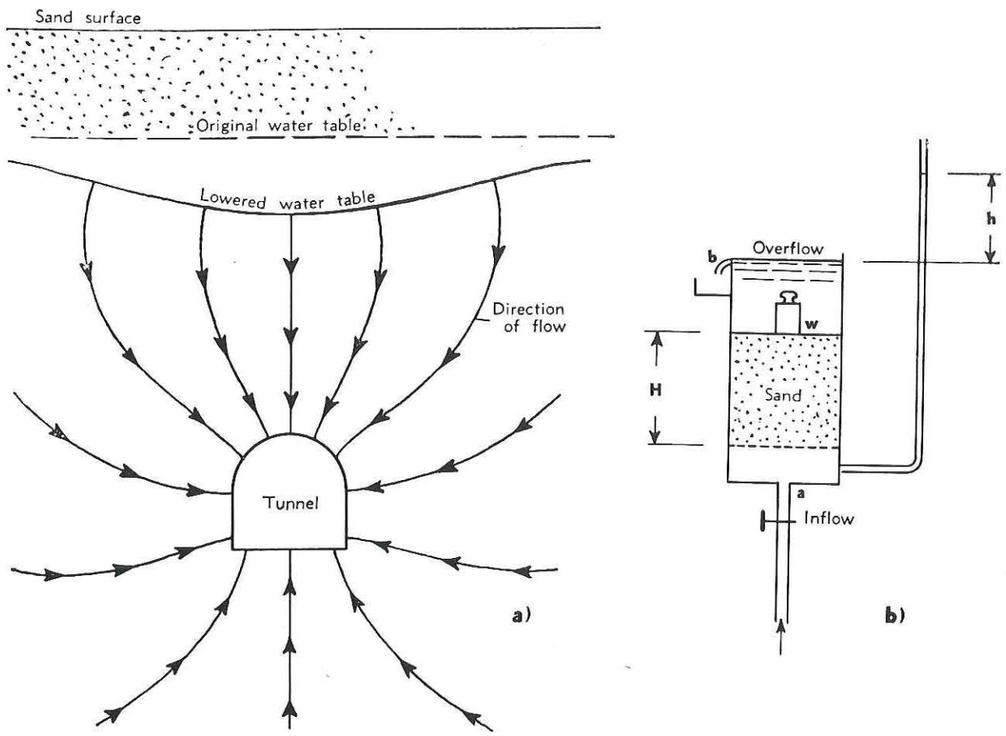


Fig. 28—Effect of seepage on tunnel floor in rock crushed to sand

a). Diagram showing flow of water from waterbearing sand into tunnel with permeable lagging. The seepage towards the floor reduces bearing capacity of sand.

b). Apparatus for demonstrating effect of rising current of seepage on bearing capacity of sand. As the head of water **h** approaches the depth of sand **H**, the weight **w** settles perceptibly indicating a loss of bearing value. When **h** becomes approximately equal to **H**, the weight sinks through the sand to the bottom of the sand layer.

loss of head **h** associated with the flow of water through the sand was measured by means of a piezometric tube. The ratio $\mathbf{i} = \mathbf{h}/\mathbf{H}$ is known as hydraulic gradient.

While the hydraulic gradient increased from zero towards unity, the structure of the sand remained practically unchanged. However, it was observed that the settlement of the weight **w** which rested on the sand increased perceptibly, indicating a decrease of the bearing capacity of the sand. As soon as the hydraulic gradient became approximately equal to unity, the sand started to boil and the weight disappeared in the sand as if the sand had turned into a liquid. The exact value of the hydraulic gradient at which this event takes place is equal to the ratio between the submerged unit weight of the sand and the unit weight of the water[1].

The hydraulic gradient at which the water enters a tunnel through the floor is commonly somewhat smaller than unity. Nevertheless, it is important enough to reduce

1. Karl Terzaghi, Erdbaumechanik, Vienna, 1925.

the bearing capacity of the sand at the floor to a small fraction of the bearing capacity of the same sand in a drained state. Hence in tunnels through cohesionless crushed rock the support of the footings of the ribs is rather difficult. Furthermore the working face in a tunnel through such material requires tight breasting. Only a small part of the face can be exposed at a time. Hence, mining must be carried out in small pockets. In other words, rock tunneling methods must be supplanted by the methods which are used when tunneling through waterbearing sand. Fortunately, crushed rock with such character is rather rare.

TUNNELS IN BLOCKY AND SEAMY ROCK

Character of rock

The term *blocky and seamy* rock indicates a rock in which the blocks located between joints are neither interconnected nor intimately interlocked. This condition is encountered in both closely jointed and badly broken rock. The joints may be narrow or wide, empty or filled with the products of rock weathering. Such a rock has essentially the character of a dense sand with very large grains and little or no cohesion. If the joints are oriented at random, the roof load is likely to be associated with a horizontal pressure on the sides of the tunnel support.

Relation between rock load, cross-section and depth of tunnel

On account of the absence of cross connections and intimate interlocking between adjoining blocks, the intensity of the load on the roof of the tunnel is determined by laws similar to those disclosed by the arching experiments with sand. According to these laws (see Eqs. 4 to 6), the load $\mathbf{H}_p$ on the roof support in tunnels at a considerable depth is independent of depth and increases in direct proportion to the sum of width **B** and height $\mathbf{H}_t$ of the tunnel. Hence if $\mathbf{H}_{p10}$ is the load on the roof of a tunnel with a width and height of 10 ft., the corresponding load on the roof of a tunnel with any width **B** feet and any height $\mathbf{H}_t$ feet through the same rock is:

$$\mathbf{H}_p = \mathbf{H}_{p10} \times \frac{\mathbf{B} + \mathbf{H}_t}{20} \qquad (8)$$

Empirical values for $\mathbf{H}_{p10}$ will be given below.

Dome action

The arch action described in the preceding article takes place when the rock located above the roof is supported only on two sides. In the immediate vicinity of the working face the rock is supported on three sides, by the rock adjoining the two sides of the tunnel and by the rock adjoining the working face. Hence in this part of the tunnel the weight of the overburden is carried not by an arch but by a half dome. A half dome can carry a heavier load than an arch with the same span. Therefore in the immediate vicinity of the working face the load on the tunnel support will be somewhat lower than at greater distances from the face.

The transition from half-dome to arch is indicated in *Fig. 29 a*. This figure represents a vertical section through the center line of a tunnel. The length l_1 is the length of tunnel blasted out per round. Immediately above the working face the overburden is carried by a half-dome, whereas at a greater distance it is carried by an arch.

Time effects and the bridge-action period

Experience shows that a mass of blocky or seamy rock does not commonly react at once to the change of stress produced by excavating a tunnel. The blast creates an

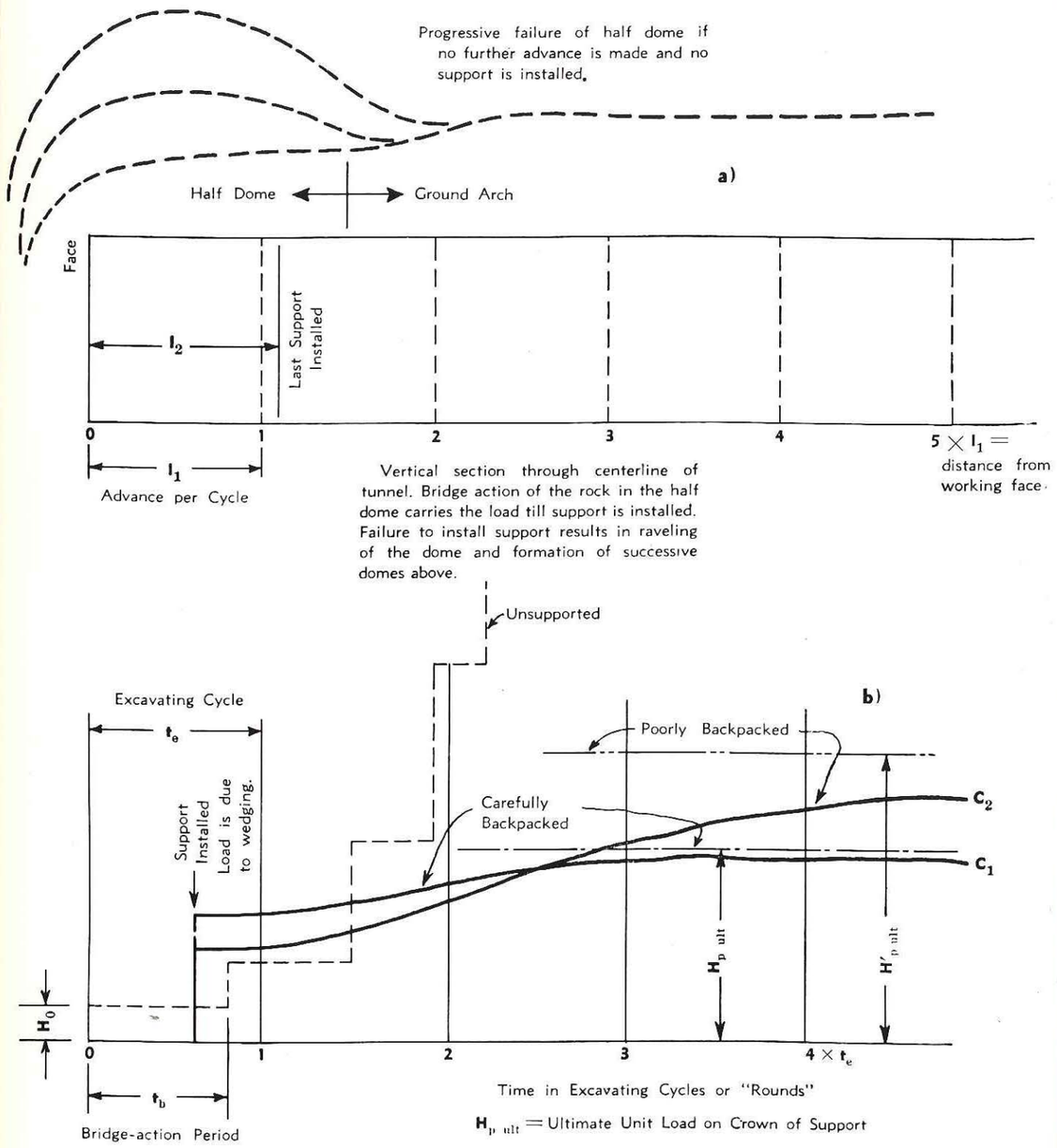


Fig. 29—Relation between time, overbreak and rock load in blocky and seamy rock.

Abscissas represent time in excavating cycles. Ordinates of dash lines indicate height **H** of overbreak in a given tunnel profile if mining is discontinued and no support is installed. The rock drops out of the roof in increasing installments and finally the heading caves in. Ordinates of full lines represent rock load H_p in feet in given profile if mining is continued and supports are installed as mining proceeds.

unsupported section of roof located between the new face and the last rib of the tunnel support. As soon as the natural roof support of this section is removed by blasting, some blocks drop out of the roof, leaving a small gap in the half-dome. If the newly exposed roof section is left without support, some more blocks drop out after a while, thus widening the gap and compromising the stability of the half-dome still more. Finally the entire mass of rock constituting the half-dome drops into the tunnel and a new half-dome is formed above the space previously occupied by its predecessor. The new half-dome also starts to disintegrate and the process continues until the tunnel section adjoining the working face is filled with rock debris.

The rate at which the progressive deterioration or raveling of the half-dome takes place depends on the shape and size of the blocks between joints, on the width of the joints, on the matrix which occupies the joints and, last but not least, on the distance l_2 between the new working face and the last support. The delays in the process of deterioration can be due to a viscous resistance of the joint-filling against rapid slippage along joints or to the progressive failure of interlocks between blocks or to both. Before the rock falls assume the character of a general breakdown of the half-dome, the blocky rock bridges the gap between the working face and the last support. Hence the time which elapses between firing the shots and the breakdown of the equilibrium of the half-dome will be designated as *bridge-action period* t_b of the rock. The value of t_b determines both the inevitable overbreak and the method for mining through the rock. The method of mining must be so chosen that the support is installed before the bridge-action period expires.

Practical importance of bridge-action period

Fig. 29 b illustrates the practical importance of the bridge-action period. In this figure the abscissas represent the time and the ordinates the vertical distances **H** between the crown of the pay line and the top of the overbreak. Time zero corresponds to the time when the shots were fired and H_0 is the height of the overbreak immediately after the shots were fired. If excavation is discontinued at that stage and the roof between the new working face and the end of the supported section of the tunnel is left unsupported indefinitely the rock will drop out of the roof in installments of increasing magnitude, as indicated by the stepped-up dash line. Between two successive rock falls indicated by steps, the roof descends imperceptibly, but it descends. The bridge action period t_b is equal to the time between firing the shot and the time when the first installment of rock drops out of the roof without provocation.

Effect of back-packing on rock load

Even if an adequate tunnel support is constructed, back packed and wedged in the freshly excavated section before the bridge-action period expires, the rock load on the roof support will increase for two reasons. First of all, as the working face advances beyond a given point, the half-dome action is superseded at that point by arch action. Second, backpacking and wedging of the support does not stop the movements of adjustment in the rock above the roof. Before the roof support was installed and wedged, all the joints in the rock above the roof opened up to some extent. This process is associated with a slight downward movement of the roof. The initial load on the roof support is equal to the force required to stop the downward movement of the lower boundary of the entire mass of blocks. However, the blocks themselves continue to change their position. The joints in the rock immediately above the roof become slightly narrower, whereas those at higher elevations open up. During this process the load on the roof support increases and it does not become constant until the movements have ceased.

The total increase of the load on the roof, and the time which elapses until the load becomes practically constant, depends to a large extent on the thoroughness with which the tunnel support is back-packed and wedged. If this is done with care, the ultimate load H_p may develop within a week after the support was constructed. This is indicated by curve C_1 in *Fig. 29 b*. The ordinates of this curve represent the load on the roof.

On the other hand, if the tunnel support was carelessly back-packed and inadequately wedged, the initial load on the support is likely to be smaller than that on the well-wedged support. However, the load will increase for many weeks, as indicated by curve C_2 in *Fig. 29 b*, and the ultimate load $H_{p\,ult}$ will be higher than $H_{p\,ult}$ because the yield of the rock towards the tunnel is associated with the progressive disintegration of the structure of the rock located within the ground arch.

Effect of span on bridge-action period

The bridge-action period for a given material increases rapidly with decreasing distance between supports. Thus for instance a very fine, moist and dense sand can bridge a space one foot wide for several hours. Yet the same sand would almost instantaneously drop through a gap between supports with a width of five feet. The shortest distance l_{2min} to which the span between the last set of roof supports and the face can be reduced is somewhat greater than the length l_1 of the tunnel section which is taken out by one round. This distance averages six-tenths of the width of the tunnel or drift. It varies considerably with the nature of the rock and seldom exceeds 15 ft. However, in moderately jointed rock it is commonly advantageous to erect the roof support at some distance l_2 from the working face. Hence if rock conditions deteriorate to such a degree that the bridge-action period t_b threatens to become shorter than the duration t_e of one excavation cycle, the support should be carried closer to the working face, or, if necessary, even tight to the face involving $l_2 = 0$.

The influence of the bridge-action period on the sequence of operations is illustrated by *Fig. 30*. This figure shows the duration of the individual cycle. If the bridge-action period is only slightly longer than the time t_v for ventilating, important

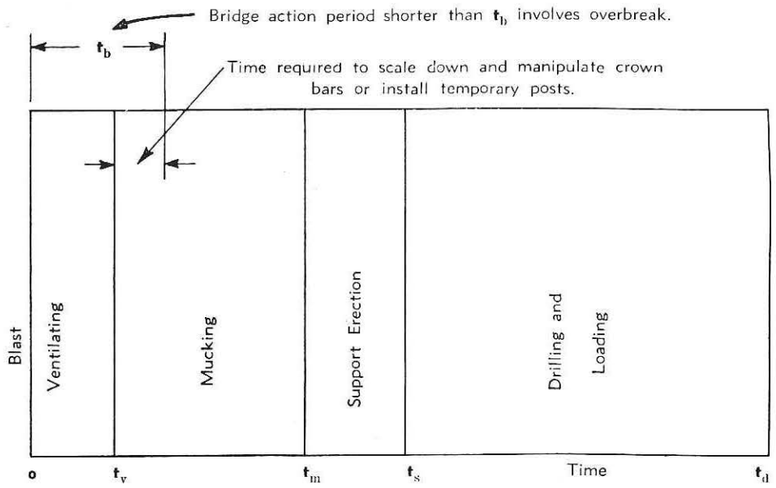


Fig.30—Diagram representing operating cycle for one round

overbreak is inevitable. If it is between t_v and t_s representing the time to the end of the support erection operation, excessive overbreak can be avoided with the assistance of crown bars or by driving a top heading so that support can be installed before mucking is complete, or both. Finally if it is as long or longer than t_s , no special precautions are required.

The bridge-action period for cohesionless sand or of badly broken rock without cementing material is almost zero. Hence if a tunnel passes abruptly from fairly sound rock into such materials, excessive overbreak at the point of transition is almost inevitable. The following incidents illustrate this statement. The tunnel shown in *Fig. 11* approached a wide seam filled with sand or completely crushed rock, and water. As soon as the rock partition between the tunnel and the seam was removed by blasting, water and sand flowed into the tunnel as shown in the illustration. In a tunnel near Philippeville in Algeria a zone of blocky and seamy quartzitic schist was encountered. The bridge-action period of this material was so short that about 60 cubic yards dropped out of the roof, leaving a dome with a height of about 30 feet above the pay line.

Roof load

No definite boundary can be established between moderately jointed and blocky and seamy rock. As a consequence the roof load may have any value between the upper limiting value for moderately jointed rock, which is $0.25\,B$, and the upper limiting value for very blocky and shattered rock, which is many times higher. The transition from one extreme to the other is gradual in some places, abrupt in others. For the sake of convenience, two degrees of blockiness will be distinguished. The first will be referred to as moderately blocky and the second one as very blocky and shattered rock.

Our knowledge of the intensity of rock loads on tunnel supports is derived chiefly from the results of tests which were carried out in various railroad tunnels in the eastern Alps. In these tests wooden blocks with known strength were inserted between the individual members of timber sets and the load H_p on the timbering was estimated from the visible manifestations of the progressive failure of the blocks. On the basis of the results of such observations the following conclusions have been reached regarding the values of H_{p10} for the rock loads on tunnel supports in moderately and very blocky rock. In wet tunnels through moderately blocky rock, the initial value of H_{p10} may be zero, and it increases to not more than about 7 ft. of rock. In wet tunnels through very blocky and shattered rock, the initial value of H_{p10} (load after one or two days) may be as high as 12 ft., and it may increase to a final value of as much as 21 ft. By introducing these values of H_{p10} into Eq. 8, we obtain for the roof load the values contained in Table 1.

TABLE 1

Rock Loads (in feet) in Blocky and Seamy Rock

	Initial value	*Ultimate value*	
Moderately blocky rock	H_p = zero	$H_{p\,ult} = 0.25\,B$ to $0.35\,(B+H_t)$	(9)
Very blocky and shattered rock	H_p = zero to $0.60\,(B+H_t)$	$H_{p\,ult} = 0.35\,(B+H_t)$ to $1.10\,(B+H_t)$	(10)

In dry tunnels the values of H_p can be very much lower than in wet tunnels. However, during spring thaws and during long wet spells, every tunnel which is not

located beneath a city with paved streets is likely to be wet. Therefore, it is advisable to disregard dry conditions.

In connection with roof load estimates, blocky and seamy rock may be considered as a crushed rock with very large grain and low porosity. Therefore, it is interesting to compare the preceding equations with those obtained on the basis of the result of the laboratory tests with sand, equations 4 to 6. Both sets of equations are assembled in Table 2.

TABLE 2

Comparison Between Rock Load (in feet) in Sand and in Blocky and Seamy Rock

Material		*Above water table*		*Below water table*[1]	
		$H_{p\ min}$	$H_{p\ max}$	$H_{p\ min}$	$H_{p\ max}$
Dense sand[2]	Initial	$0.27\ (B + H_t)$	$0.60\ (B + H_t)$	$0.54\ (B + H_t)$	$1.20\ (B + H_t)$
	Ultimate	$0.31\ (B + H_t)$	$0.69\ (B + H_t)$	$0.62\ (B + H_t)$	$1.38\ (B + H_t)$
Loose sand[2]	Initial	$0.47\ (B + H_t)$	$0.60\ (B + H_t)$	$0.94\ (B + H_t)$	$1.20\ (B + H_t)$
	Ultimate	$0.54\ (B + H_t)$	$0.69\ (B + H_t)$	$1.08\ (B + H_t)$	$1.38\ (B + H_t)$
Moderately blocky[3]		$H_{p\ in} =$	0	increasing up to $H_{p\ ult} = 0.35\ (B + H_t)$	
Very blocky and shattered		$H_{p\ in} = .60\ (B + H_t)$		increasing up to $H_{p\ ult} = 1.10\ (B + H_t)$	

1. Values are roughly equal to twice those for dry sand.
2. Values computed on basis of laboratory tests.
3. Values computed on the basis of the results of observations in railroad tunnels.

The values for rock tunnels given in Table 2 were obtained from observations in tunnels below the water table. Therefore they should be compared with those for cohesionless sand below the water table. The table shows that the ultimate rock load in feet of rock in moderately blocky and seamy rock is considerably lower than the ultimate minimum value for dense sand whereas the corresponding value for very blocky and shattered rock is roughly equal to the ultimate minimum value for loose sand. These data demonstrate that the inevitable yield of the rock towards the tunnel, prior to wedging and backpacking, is important enough to develop the arch action to the fullest extent. Any yield in excess of this amount, due to careless mining or inadequate backpacking would probably increase the ultimate rock load. According to the table, the initial rock load on the roof support in blocky and seamy rock is very much smaller than in sand, everything else being equal, but the ultimate rock load is of the same order of magnitude.

Roof pressure in tunnels above the water table

If the joints in a blocky and seamy rock do not contain clay, the pressure of the rock on the tunnel support may be as high as one-half of the pressure exerted by the same rock on the same tunnel at a considerable depth below the water table. On the other hand, if the joints are partially or entirely filled with clay, a nominal support may be sufficient to hold up the roof during the dry season because in a dried-out state the clay acts as a cementing material. However, during long wet spells the clay ceases to act as an effective binder whereupon the pressure on the tunnel support becomes as heavy as if the joints were lubricated.

Owing to this cause, several large tunnels, (including the Stapleton Tunnel in England and The Altenbeker Tunnel in Germany) which were mined and timbered during the dry season, caved in soon after the autumn rains started. Hence if it is not certain

that the rock located above the tunnel will remain practically dry throughout the year, it is advisable to design the tunnel support on the basis of the values obtained by means of Eqs. 9 and 10, regardless of the appearance of the rock during mining operations.

EARTH PRESSURE PHENOMENA IN DECOMPOSED ROCK AND IN CLAY

Relationship between decomposed rock and clay

Chemical alteration changes many rocks including all igneous rocks and most shales and schists into clay. Some rocks may be converted entirely into clay whereas in others the transformation is limited to some of the mineral constituents. Alteration may take place throughout the entire mass or only along fissures. In any event the properties of the altered rock are entirely different from those of the original rock and are commonly similar to or even identical with, those of clay.

Since large bodies of chemically altered rock are not uncommon, some tunnels are located partly or entirely in such rock. A knowledge of tunneling conditions in altered rock is therefore of greatest importance to the tunnel engineer.

The processes of excavating and of installing a tunnel support induce arching in decomposed rock as they do in blocky and seamy or in crushed rock. In other words, the ultimate rock load is commonly much smaller than the weight of the overburden. However, the development of the ground arch in decomposed rock is associated with and followed by phenomena which are wholly absent in tunnels through shattered or crushed, but chemically intact, rock. The bridge-action period in decomposed rock is very much longer than in crushed rocks. Therefore breasting is rarely necessary in decomposed rock. On the other hand, the rock load on the tunnel support is likely to increase, in the course of weeks or even months, to a value which is many times higher than the initial one. Very similar phenomena are encountered when tunneling through sedimentary clay. As a matter of fact, tunneling conditions in decomposed rock are so similar to those in clay that the methods for tunneling through clay can be used without any modification on tunnel jobs in decomposed rock.

The striking resemblance between tunneling conditions in decomposed rock and in clay is due to the low permeability, the high compressibility and other peculiar properties of clayey materials in general. In order to be able to grasp the practical implications of the clay content of decomposed rock and to take full advantage of clay tunneling experience when tunneling through altered rock, the engineer must be familiar with the significant properties of clay in general. The following paragraphs contain a summary of our present knowledge of clays and of clay behavior on tunnel jobs.

Difference between sand and clay

Both sand and clay constitute aggregates of mineral particles which can be separated from each other by agitating the aggregate with water. The difference between the two materials resides chiefly in the size and shape of the particles.

The term *sand* is commonly applied to aggregates of more or less equi-dimensional grains greater than 0.05 mm. or about 1/500-inch, whereas clay owes its peculiar properties to what are known as clay minerals with a grain-size of less than 0.002 mm. or about 1/12500-inch.

Clay minerals are subdivided into three groups, the Kaolin, Illite and Montmorillonite group. Most of the clay constituents have the appearance of minute mica flakes, but their physical properties are very different. Two sands with equal grain-size are very

similar to each other, whereas two clay fractions with equal grain-size may have very little in common. The worst troublemakers among the clay minerals are the members of the Montmorillonite group, because the presence of a high percentage of these minerals in a clay is commonly associated with an excessive swelling tendency.

In a finely subdivided state the micaceous constituents of many rocks, such as chlorite-, sericite- and mica-schists and various shales, possess all the properties of real clays. Hence if a rock containing a high percentage of such minerals is completely crushed, for instance along the walls of a fault, it acquires even in a chemically unaltered state the physical properties of a clay soil.

In connection with tunneling, the most important physical properties of clay are the swelling associated with the removal of a pressure, the relation between pressure and shearing resistance, and the rate at which the clay reacts to a change in the stress conditions.

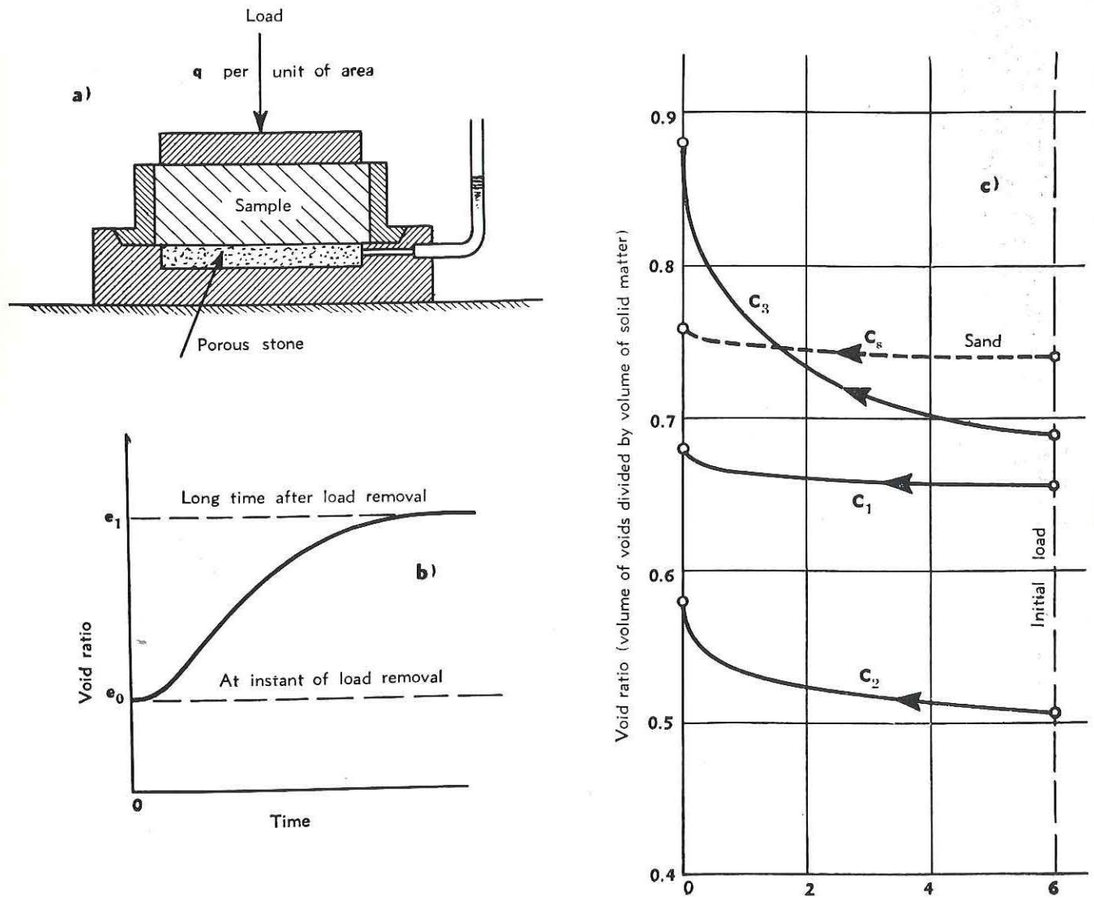


Fig. 31—Volume, time, pre-load relationship of clays

Swelling due to load removal

The effect of a load reduction or removal on the water content of a clay can be investigated by means of the consolidation apparatus shown in *Fig. 31 a*. It consists of a low, cylindrical vessel and a loading device. The bottom of the vessel is covered with a porous stone. The voids of the stone are filled with water which communicates with the water contained in a short, open piece of pipe. In order to get information on the swelling properties of a clay at a given depth below the surface, an undisturbed sample of the clay is secured at that depth. It is trimmed, introduced into the container of the consolidation apparatus and charged with a unit load **q** equal to the overburden pressure per unit of area. After its water content has become constant, the load is removed by increments. After each load removal the clay swells at a decreasing rate as shown in *Fig. 31 b*. In this figure the abscissas represent time and the ordinates the corresponding void ratio. The void ratio is equal to the volume of the water divided by the volume of the solid matter. Since the increase of the volume of the clay is exclusively due to the increase of its water content, the increase of the ordinates in *Fig. 31 b* also represents the increase of the volume of the clay due to swelling. The time which elapses until the clay practically ceases to swell depends on the permeability of the clay, everything else being equal. The reason for this relationship will be explained below.

After the swelling due to the removal of a load increment has ceased, the void ratio is determined, and the next load increment is removed. By plotting these void ratios against the unit load under which the clay expanded, a curve similar to the swelling curves C_1 to C_3, *Fig. 31 c* is obtained. Curve C_1 represents a greenish, calcareous clay from the Atlantic coast of the United States. Curve C_2 shows the results of tests on a clay sample from a subway tunnel in Chicago, and C_3 those of tests on an energetically swelling clay encountered in the construction of a subway tunnel in the subrubs of Paris, France. For comparison the swelling curve C_s for an ordinary sand has been added.

For any given material, sand or clay, the increase of the void ratio due to the removal of a load increases with the intensity of the load under which the clay had previously been consolidated. Part of the excessive swelling of the clay represented by curve C_2 was due to the fact that this clay has been consolidated under the influence of a pressure far in excess of the present overburden pressure and part of it to a high Montmorillonite content.

When excavating a tunnel through clay, the clay adjoining the tunnel walls passes through the same process as that represented by the curves C_1 to C_3 in *Fig. 31 c*. The water required to produce the swelling is drawn out of the clay located at a greater distance from the tunnel. It is associated with a softening-up of the clay at the tunnel walls.

It has often been claimed that the softening and swelling of stiff clays in tunnels is due to the contact of the clay with the moist atmosphere in the tunnel. The following observations show that this opinion is unjustified.

In the tunnel through the clay represented by curve C_3 in *Fig. 31 c* it was found that the average water content of the clay adjoining the walls of the tunnel increased within about two weeks from an average of 56 per cent to more than 100 per cent of dry weight. The increase of the water content was associated with a heavy swelling pressure sufficient to crush the timbering. It was claimed that the water came out of the air in the tunnel. In order to find whether this explanation was correct, a sample of clay was taken at the heading, placed in a dish and exposed to the atmosphere in

the tunnel. In a few days the sample was dry. Hence it was obvious that the water which entered the clay adjoining the walls did not come out of the air but out of the clay located beyond the zone of swelling. As a matter of fact, subsequent horizontal borings and water content determinations showed that the water content of the clay at a distance of more than about 15 ft. from the tunnel was well below the average water content of the clay prior to construction.

Relation between pressure and shearing resistance

The relation between pressure and shearing resistance for particle aggregates such as sand or clay is commonly investigated by means of the shear box apparatus shown in *Fig. 32 a*. The sample is introduced into a square box. Its top surface is covered with a plate and loaded. The bottom of the box is covered with a porous stone. The voids of the stone communicate with a short, open pipe.

The lower part of the box is stationary, whereas the upper one can be pulled in a horizontal direction. Pulling the top part ultimately produces a shear failure in the sample along the plane located between the lower and upper part of the frame.

Fig. 32 b represents the results of shear tests on a sample of sand. The abscissas represent the unit load on the sample and the ordinates, the shearing force per unit of area of the shear plane at the instant of failure. At any given unit load **q** the corresponding unit shearing resistance **s** is equal to **q** tanø. The angle ø is known as *angle of internal friction*. The value of ø for sand is practically independent of whether or not the test load was preceded by a higher one. It is also practically independent of the rate at which the shearing force is applied. For a given sand the value ø increases considerably with increasing density. It ranges for sands in a loose state

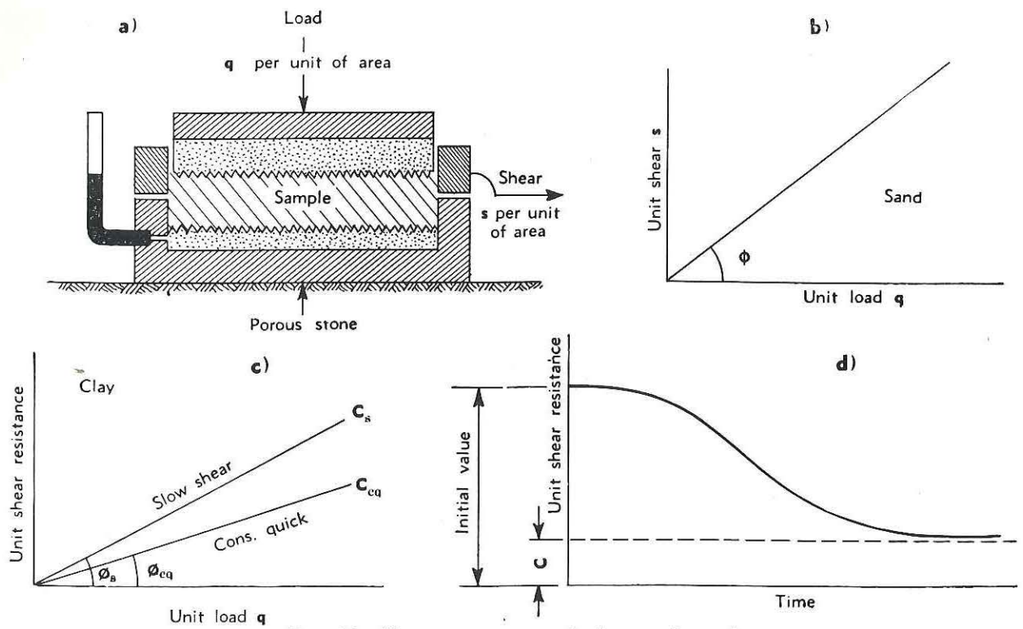


Fig. 32—Shear resistance of clay and sand

between 30° and 34° and in a dense state between 35° and 46°. Quite exceptionally values as low as 28° have been obtained for loose sand and values of more than 46° for dense ones.

Fig. 32 c represents the results of shear tests on a clay which was introduced into the shear box in a very soft and completely saturated state. In contrast to sand, clay is very compressible and an increase of the load is associated with a considerable decrease of the water content. The specimen is loaded and the shear test is made after all the excess water has drained out of the clay through the voids of the porous stone on the bottom of the shear box, *Fig. 32 a*. On account of the low permeability of the clay the process of drainage requires from several hours to several days.

While the shearing force is applied the water content of the clay further decreases very considerably. In order to provide for this supplementary drainage the shearing force must be increased very slowly. Such a test is called a *slow shear test. The* results of a series of slow shear tests are represented by the straight line C_s in *Fig. 32 c*. It rises from the origin at an angle ϕ_s to the horizontal. The angle ϕ_s ranges between 28° and 30°, exceptionally as low as 20°.

If the shearing force is so rapidly applied that the water content of the clay remains practically unchanged, the line C_{eq} is obtained. Since the rapid application of the shearing force is preceded by a complete consolidation of the clay under the vertical load, tests of this type are known as *consolidated-quick shear tests*. The slope angle ϕ_{eq} of the consolidated-quick shear line C_{eq} is always very much smaller than the "slow" value ϕ_s for the same clay. It ranges between 14° and 20° and is exceptionally as low as 12°.

Cohesion of clays

The tests illustrated by *Fig. 32 c* were made on a clay which was very soft to start with. The cohesion of clay in such a state is negligible. Therefore the load-shear lines pass through the origin. The relation between load and shearing resistance represented by these lines differs from the corresponding relation for sand, *Fig. 32 b*, only inasmuch as the slope angle of the load-shear lines is smaller and the resistance against shear develops very gradually.

In contrast to soft clay on which the shear tests were made, clay specimens cut out of natural clay strata commonly possess many of the properties of solid, somewhat brittle materials. The strength of such clays is measured by the greatest unit load which unconfined cylindrical specimens of the clay can sustain. This unit load will be referred to as *unconfined compressive strength*, q_u. The following are representative values:

	Very Soft	*Soft*	*Medium*	*Stiff*	*Very Stiff*	*Hard*
q_u (Tons per sq. ft.)	0.25	0.25-0.5	0.5-1.0	1.0-2.0	2.0-4.0	4.0

The shearing resistance of the clay in an unconfined state is commonly known as *cohesion*, **c**. It is roughly equal to one-half of the unconfined compressive strength q_u.

By kneading and mixing a stiff clay with a sufficient quantity of water it can be transformed into a soft clay similar to the one to which the diagram *Fig. 32 c* refers. Thus the cohesion of the clay becomes negligible. However, by squeezing the water out of the clay again, the major part of its original cohesion can be recovered.

Squeeze and rate of softening

The process of excavating a tunnel in clay is associated with a decrease of the pressure in the clay adjoining the heading. The greatest drop in pressure takes place in the direction towards the heading. Therefore the clay slowly advances towards the heading from top, sides and bottom. If mining is discontinued the working face slowly advances into the tunnel unless it is bulkheaded. This process is known as *squeeze*.

The decrease of the pressure in the clay is associated with at least a small increase of the water content and a decrease of the shearing resistance. Both processes combined produce the impression that the clay becomes softer.

Since any change of the water content of a clay proceeds very slowly, even a sudden removal of the pressure on a clay produces no more than a slow decrease of the shearing resistance of the clay as shown in *Fig. 32 d*. In this figure the abscissas represent the time and the ordinates the unit shearing resistance at different times after the load on the clay was suddenly removed. For a few hours or even a few days the shearing resistance seems to remain almost unchanged. Then it decreases quite rapidly, continues to decrease at a lessening rate and finally approaches a constant value **c** which represents the ultimate cohesion.

Mechanics of time effects in clay

In the preceding paragraphs it was demonstrated on the basis of test results that the effects of a change of pressure on the water content and the shearing resistance of clay takes place very slowly. An investigation of the causes of the observed phenomena has shown that the time lags are chiefly due to the high compressibility of the clays combined with their low permeability. The quantity of water which drains out of a porous body with given dimensions after the application of a given load depends only on the compressibility of the material. On the other hand the time it takes until most of the water is squeezed out depends on the permeability. The lower the permeability the

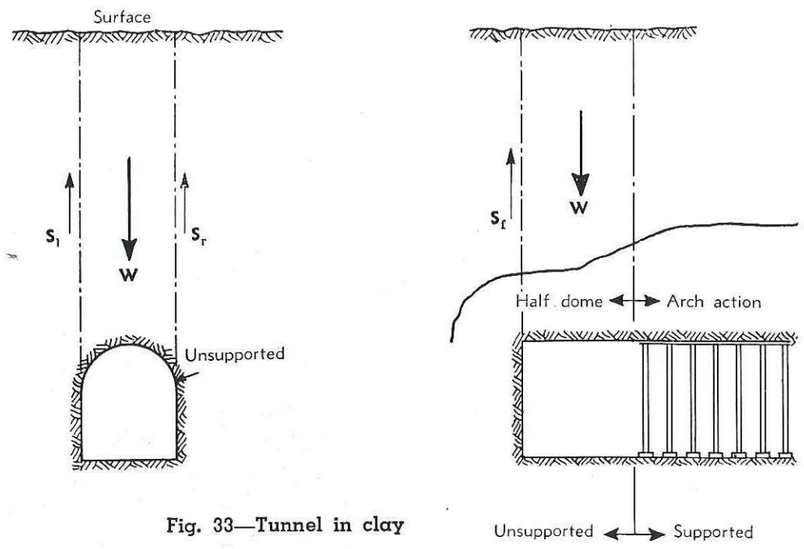


Fig. 33—Tunnel in clay

more time is required for this process. The same statement applies to the rate of increase of the water content due to the reduction of a load.

Dome action and rate of squeeze in clay tunnels

Fig. 33 is a vertical section through the unsupported part of a tunnel through clay. The fact that the roof does not need any support indicates that the entire weight **W** of the clay is transferred by half-dome action onto the clay located at some distance from the heading. The weight transfer requires that the sum of the shearing forces $\mathbf{S}_l$ plus $\mathbf{S}_r$ plus $\mathbf{S}_f$ on vertical sections through the sides and face of the heading be equal to the weight **W**.

On account of the physical properties of clay described above, the rapid change in stress in the clay due to the excavation of the heading is followed by slow movements associated with a gradual increase of the water content of the clay adjoining the tunnel walls. All these movements take place in the directions of least resistance, all of which are directed towards the heading. They simply indicate that the clay has not yet adjusted itself to the change in stress produced by the tunnel excavation.

The rate at which the roof and the walls of the heading advance into the tunnel will be referred to as *rate of squeeze*. At a given depth in a given clay the rate of squeeze increases rapidly with increasing dimensions of the unsupported part of the tunnel. Hence by reducing these dimensions one can reduce the rate of squeeze in any clay to an amount compatible with construction requirements.

In very soft, soupy material, such as river silt, the squeeze can be sufficiently reduced only by reducing the unsupported part of the tunnel to small pockets. However, even in what is commonly considered as soft clay, the length of the unsupported section of tunnels with a width and height of eight feet can be increased to eight or ten feet without producing a noticeable squeeze. The rate of squeeze in such headings can only be determined by repeatedly measuring the distance between reference points attached to the working face, the walls, the roof and the floor.

Ground cylinder

After the heading has advanced to a certain distance beyond a given station in the tunnel the half-dome action which diverts the major part of the overburden load away from the roof is superseded by arch action. At the same time the clay squeezes towards the side and the bottom of the tunnel. The squeeze is associated with a deformation involving a lengthening of every block of clay in radial and a shortening in circumferential directions. This deformation brings both the internal friction and the cohesion into play, because neither one of these resistances are active until movement towards the tunnel occurs. Once the strength of the clay is mobilized beneath the tunnel on account of a sufficient amount of squeeze into the tunnel the clay beneath the tunnel constitutes an inverted ground arch. In a similar manner the clay which squeezes into the tunnel from the sides acquires the properties of a side arch. The roof arch, the side arches and the bottom arch merge into what may be called a *ground cylinder* which carries the major part of the pressure in the clay located beyond the ground cylinder.

Load increase and load-increase period

As soon as a tunnel support is placed and wedged, the squeeze is practically stopped, although the clay has not yet adjusted itself to the changes produced by the

tunnel excavation. As a consequence the clay pressure on the support increases. This load increase due to stopping the squeeze can be demonstrated by the following laboratory experiment.

A sample of clay is introduced into the consolidation apparatus shown in *Fig. 31 a* and consolidated under a load q_0 which reduces its water content to w_0. Then the load is suddenly removed, whereupon its thickness increases due to swelling and the top plate goes up until the top plate is restrained, at which time upward pressure starts to develop. In *Fig. 34 a* the abscissas represent the time and the ordinates the upward pressure of the clay. At time zero the load on the clay was reduced from q_0 to almost zero and then it was kept at this value. In *Fig. 34 b* the abscissas also represent the time and the ordinates the water content.

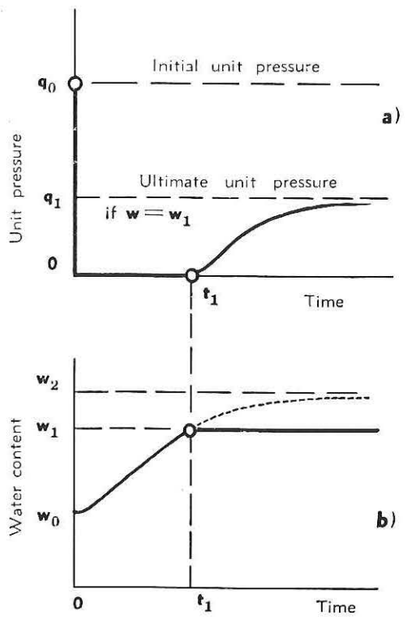


Fig. 34—Swelling, pressure, time effects of clay

At time t_1 the water content of the clay is w_1. If at that time the further rise of the top cover of the clay sample is prevented, the water content of the clay ceases to increase, but in exchange the pressure on the cover plate increases and approaches a value q_1 intermediate between zero and q_0 as indicated in *Fig. 34 a*. Similar phenomena can be observed when making a shear test. If the shearing force is kept constant, the displacement by sliding continues at a constant rate. If this movement is stopped, the shearing stresses on the surface of sliding decrease and approach a constant value considerably lower than the initial one.

On account of such time effects, the load on tunnel supports always increases very considerably though at a decreasing rate. Experience indicates that the load-increase period ranges between several weeks and many months.

Load-increase in clays with high swelling capacity

It was mentioned before that the swelling of a clay due to removal of a load depends not only on the nature of the clay but also on the intensity of the pressure under which the clay was consolidated.

If a natural clay stratum was consolidated by the weight of thick soil strata which were subsequently removed by erosion, the horizontal pressure in the clay is likely to remain forever considerably higher than the pressure exerted by the remaining strata. Such clays are known as *pre-loaded clays*. On account of their high density, their permeability is commonly very low. Hence if a pre-loaded clay has a high swelling capacity, the rate of squeeze is very low and the increase of the clay pressure on tunnel supports increases very slowly. However, at moderate depth below the surface, the ultimate value of the clay pressure on the tunnel support can ultimately exceed the present overburden pressure.

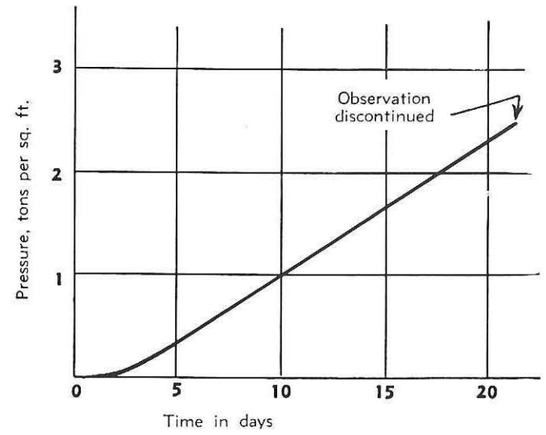


Fig. 35—Time, pressure curve of a clay in France

The preceding conclusions are illustrated by the results of the following observations. In Provins, east of Paris, France, a stiff, swelling clay is mined for commercial purposes. The stratum of clay has a thickness of about 115 ft. It is covered by about 65 ft. of water-bearing sand and silt. The galleries have a square cross section of 6 by 6 ft. and they are located at an average depth of 110 ft. below the surface. At that depth the overburden pressure is roughly 7 tons per sq. ft. The roof, sides, and bottom of the tunnel are braced with 10-in round hardwood timbers without any gaps between them. Immediately after mining the clay stands without support.

After about a week the swelling starts. After about three months the lining timbers are crushed and require replacement. During this period the water content of the clay adjoining the tunnel increases from its initial value of about 35 per cent, to about 70 per cent of the dryweight. It was estimated that the pressure required to crush the timbering is about 25 tons per sq. ft., which is more than three times the overburden pressure. This observation indicates that the horizontal pressure in the clay is very much greater than the corresponding overburden pressure.

In order to get an accurate conception of the rate of increase of the pressure, an experimental gallery with a length of about 300 ft. was constructed in untouched clay. It was lined with square, hardwood timbers, placed side by side and carefully jointed together. At one point a horizontal hole with a diameter of 12 in. was drilled through the timber lining and the adjoining clay. A pressure cell was installed in the hole, at a distance of about 5 ft. from the wall, which registered the pressure exerted by the clay upon the cell. *Fig. 35* represents the results of the readings during the first twenty days. During the following three months the pressure increased to about 15 tons per sq. ft. At that time, the readings had to be discontinued on account of labor troubles, but the rate of pressure-increase was still constant. (Data obtained in May, 1939, by courtesy of Mr. K. Langer, Consultant to the Laboratoroires du Batiment et des Travaux Publics, Paris, who designed and installed the cell and made the readings).

Slaking at tunnel walls and the bridge-action period for clay

Many stiff clays contain an intricate network of closely spaced hair cracks. If an unconfined cylindrical specimen of such a clay is loaded, it disintegrates into small angular fragments as soon as its length has been slightly reduced by the pressure. The disintegration is due to the opening up of joints between fragments.

In tunnels the squeeze produces similar deformations and, as a consequence, a similar disintegration. The deformation is illustrated by *Fig. 36*, which represents a cross-section of a circular

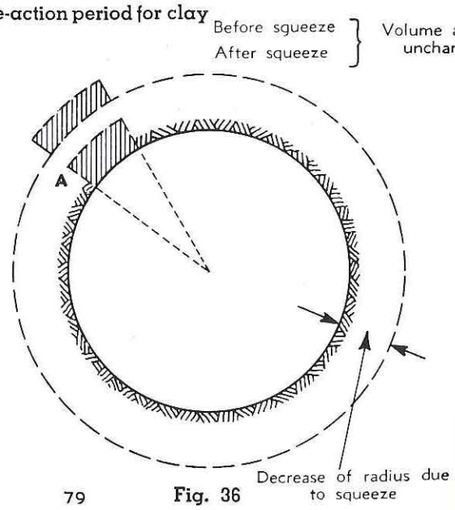


Fig. 36

tunnel. The squeeze reduces the length of the periphery of the circular section. Hence the length of every block of clay, such as **A**, measured parallel to the walls, decreases whereas its width measured at right angles to the wall increases. As soon as the percentage shortening exceeds a certain very small value, disintegration begins. This process is commonly known as *slaking*. It is often erroneously ascribed to weathering due to the exposure of the clay to the air. It is usually nothing but one of several manifestations of squeeze.

As soon as the clay above the roof starts to slake it loses what little strength it had to begin with, whereupon masses of clay start to drop out of the roof. The time which elapses until the roof disintegration starts represents the bridge-action period for the clay. It ranges between several hours and several days. The roof support must be installed before the bridge-action period expires.

In connection with soft clays the term bridge-action period has no meaning, because soft clays do not disintegrate. They merely squeeze.

TUNNELS IN SQUEEZING OR SWELLING ROCK

Properties of squeezing rock

Squeezing rock is merely rock which contains a considerable amount of clay. The clay may have been present originally, as in some shales, or it may be an alteration product. The rock may be mechanically intact, jointed, or crushed. The clay fraction of the rock may be dominated by the inoffensive members of the Kaolinite group or it may have the vicious properties of the Montmorillonites. Therefore the properties of squeezing rock may vary within as wide a range as those of clay.

Crushing combined with chemical alteration may produce a material comparable to soft river silt whereas the pressure phenomena in some decomposed igneous rocks and schists are similar to those encountered in stiff, swelling clay. If a squeezing rock is so soft that it does not require blasting, the same methods must be used which are commonly employed when tunneling through clay with a similar consistency. These methods will be described in detail in a companion volume dealing with *"Earth Tunneling with Steel Supports."* When tunneling through decomposed rocks the construction difficulties commonly increase with increasing swelling capacity of the rock.

Tunnels in squeezing rock with moderate swelling capacity

The few pressure observations which have been made in tunnels through rocks of this type indicate that the relation between the roof load H_p and the cross section of such tunnels is similar to the relation expressed by Eq. 8, repeated here,

$$H_p = H_{p10}\,\frac{B + H_t}{20}$$

which was established for blocky and seamy rock.

The value H_{p10} increases at least for several weeks after excavation and it also seems to increase to some extent with increasing depth of the tunnel below the surface. The highest pressures which were observed at depth of several hundred feet indicate that the corresponding value of H_{p10} increased from an initial value of about 23 ft. to a final one of about 42 ft. At depths of more than a thousand ft. the initial value of H_p can be as high as 30 ft. It increases within the first months up to about 70 ft. of rock.

At any depth the rock tends to squeeze into the tunnel from all sides. Experience indicates that the ratio between the unit pressure on the sides and the roof is roughly equal to one-third and the ratio between the pressure on the floor support and the roof support is about one-half.

Slaking phenomena and the bridge-action period

In a decomposed state many igneous rocks and almost every schist and shale contain a dense network of hair cracks. In such rocks the squeeze is inevitably associated with a gradual disintegration of the rock at the tunnel walls. In the preceding article this process has been referred to as "slaking." The cause of slaking is illustrated by *Fig. 36*. Every rock which slakes has a definite bridge-action period, because slaking leads sooner or later to masses of slaked rocks dropping out of unsupported sections of the roof. At given dimensions of these sections the bridge-action period depends chiefly on the rate of squeeze and the spacing of hair cracks. In a given rock the bridge-action period can be increased by reducing the length of the unsupported roof section.

Slaking is commonly associated with an increase of the moisture content of the rock at the tunnel walls. The moisture may accelerate the process of slaking. However, both the slaking and the increase of the water content are not the cause but the consequence of the squeeze. Some of the water may be due to condensation on account of the low temperature at greater distances from the tunnel. Therefore by spraying the exposed rock surfaces with bituminous material, gunite, or shotcrete, the process of slaking may in some instances be retarded, but there is no evidence that this method ever stopped a squeeze.

The following observations illustrate the visible effect of disintegration due to squeeze in wet tunnels. In a railroad tunnel, located in slaty rock, the rock appeared dry and almost intact after blasting. However, within 24 hours after blasting the walls became very moist; hair cracks became visible between the laminae of which the rock consisted, and within the next few days the rock assumed the character of a soft, plastic material which exerted heavy pressure on the tunnel support. Finally it became so soft that it squeezed through the joints in the lagging.

Time effects in tunnels through swelling rock

The term *swelling rock* refers to rocks the squeeze of which is chiefly due to swelling. Swelling rocks are always at least moderately dense, having the consistency of stiff or hard, pre-loaded clays.

A graphic description of the behavior of such rock in a tunnel was published by T. S. Lovering.[1] It refers to a section of the Moffat Tunnel in which decomposed granite of the swelling type was encountered. "In this section the ground swelled continuously and irresistibly. In the water tunnel, a bore parallel to the railroad tunnel, 12x18-inch timbers of Oregon fir were broken like matchsticks. . ." "On the eastern side of the fault zone the walls are equally deceptive, very few gouge seams are present, and the rock seems to be bleached but sound granite. It is quite surprising to observe material of this appearance slowly closing up the tunnel without developing any cracks or shear planes. In fact, the behavior of the ground so impresses hard rock miners as being uncanny that they will seldom work in this part of the tunnel for more than a few days before quitting."

The bridge-action period of swelling rocks depends on the same factors as that for ordinary squeezing rock. In swelling rock with a long bridge-action period, such as the decomposed granite described by Lovering, the initial load on the tunnel support is almost exclusively due to wedging. However the load increase period may amount to many months and during this period the pressure may become heavy enough to crush even a very heavy tunnel support.

The failure of the tunnel support is associated with an almost instantaneous relaxation of the pressure. This is impressively demonstrated by the aspect of the tunnel after failure. Although the strength of the split, crushed or twisted timbers is much smaller than the strength of the same timbers in an intact state, these timbers are commonly strong enough to sustain for at least several days the pressure after failure.

1. T. S. Lovering, Geology of the Moffat Tunnel, Colorado, Trans. Am. Inst. Min. & Met. Eng., Vol. 76 (1928) pp. 337 to 345.

If a new tunnel support is constructed the pressure again increases, but the swelling associated with the failure of its predecessor permanently reduces the ultimate intensity of the pressure.

The squeeze of a swelling rock into a tunnel is always associated with an increase of the water content and a loss of strength of the rock adjoining the tunnel. Therefore, it is a common occurrence that the footings of the ribs which were originally capable of sustaining the pressure, gradually penetrate the material on which they rest. At the same time the bottom heaves and displaces the track and the conduits in the tunnel. Therefore, it is advisable to provide the tunnel support in swelling rock with circular ribs. The circular form imparts to steel ribs the greatest strength per pound of steel employed and makes them equally resistant to pressure from all directions.

Rock pressure in swelling rock

The pressure on the support in tunnels through swelling rock depends primarily on the swelling capacity of the rock which is analogous to the swelling capacity of clays (see *Fig. 31 c*). Therefore no general rules comparable to the rule expressed by Eq. 8 for the influence of the width and height of the tunnel on the rock load can be established. As a matter of fact it is not even known whether the pressure varies with the width of the tunnel. Therefore information on the pressure exerted by swelling rock can be obtained only from observations in tunnels and data of this kind are still very scarce.

In shallow tunnels the ultimate pressure on the tunnel lining may be considerably higher than the overburden pressure. This opinion is based on extrapolation from experience in clay tunnels such as those referred to in the preceding article.

In deep tunnels through swelling rock, pressures of 10 tons per sq. ft. are not uncommon. Exceptionally, pressures as high as 20 tons per sq. ft. have been encountered. A pressure of 20 tons per sq. ft. is equivalent to the weight of a layer of rock with a thickness of not more than about 270 ft. This fact demonstrates that the process of swelling does not interfere with the development of a ground cylinder which carries the major part of the overburden pressure.

Provisions for expansion in swelling rock

The mechanics of the process of swelling illustrated by *Fig. 34* indicate that the expansion of swelling material produces a permanent reduction of the ultimate value of the swelling pressure on a tunnel support. Experience in tunneling through swelling rock confirms this conclusion. Hence when the first railroad tunnels through swelling rock were built, in the middle of the last century in England, it was decided to reduce the pressure on the permanent lining by providing a clearance of six inches between the rock and the outer face of the masonry. This measure proved to be successful and it has often been used.

Quite recently, in the construction of a double-track subway tunnel through the swelling clay represented by curve C_3 in *Fig. 31 c* in Paris (France) the same clearance was provided.

Yielding tunnel supports in swelling rock

In mining through swelling clay or rock it has been customary to construct a heavy set to start with, and if it is crushed, to replace it by a new and stronger one. This procedure permits the rock to squeeze into the tunnel through a distance of at least six inches prior to the construction of the permanent lining. As a consequence, it

produces a very substantial reduction of the ultimate load on the tunnel support. However, it is doubtful whether this is economical and does not furnish any definite information concerning the pressure which will act on the permanent lining.

In order to avoid the complications and inconveniences associated with cleaning up the tunnel after each failure of a lining, the following procedure would deserve a trial. Instead of constructing a tunnel support which either remains intact or fails completely, the support should be provided with ribs which are strong enough to stand up under the pressure while the rock squeezes into the spaces between the ribs. It is obvious that the pressure per unit of area on the outer flanges of the ribs cannot exceed the bearing capacity q_d of the swelling rock. Any pressure in excess of q_d causes the rock to flow around the outer flange of the rib as indicated in *Fig. 37*. The resistance q_d against penetration of the rib into the decomposed rock can be determined by means of a simple test in the tunnel.

The space between the ribs is bridged by a lagging which can fail under the rock pressure without injury to the ribs. If it fails, the material which squeezes into the space between the ribs is excavated and the lagging is replaced. It is much cheaper to replace the lagging several times than to replace an entire crushed support once. After the pressure in the rock surrounding the tunnel has been sufficiently relieved to establish ground cylinder action, the space between the ribs is excavated and the concrete for the permanent lining is poured. The ribs are left in place and serve as reinforcement.

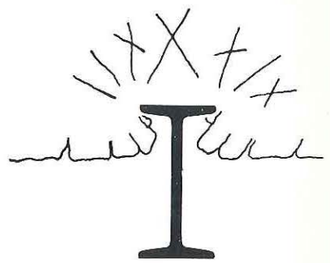


Fig. 37—Flow of swelling rock around tunnel rib

Another possibility would be to insert into the ribs compressible spacers which permit the diameter of the ribs to decrease very considerably without injury to the ribs.

In order to get quantitative information on the time-pressure characteristics of the rock and on the pressure-relieving effect of the squeeze, sturdy pressure measuring devices should be inserted into some of the ribs, by means of which the radial pressure on the ribs can be determined periodically.

Criteria for swelling rock

Throughout the preceding text great emphasis has been placed on the beneficial effects of careful back-packing and wedging. In moderately jointed or blocky and seamy rock, careful back-packing considerably reduces the load increase period and the ultimate pressure on the tunnel support. Even in squeezing rock with a low swelling capacity the presence of empty spaces between support and rock is likely to do more harm than good. Providing an opportunity for the rock to squeeze into the tunnel should only be used as a last resort, if the tendency of the rock to swell is very conspicuous. Hence, before a rock is classified as a swelling rock, its swelling properties should be carefully investigated. If the volume of samples of the freshly exposed rock or of samples from a freshly recovered core does not increase by at least 2 per cent during immersion in water, the rock cannot be classified as swelling rock and the tunnel support should be tightly wedged.

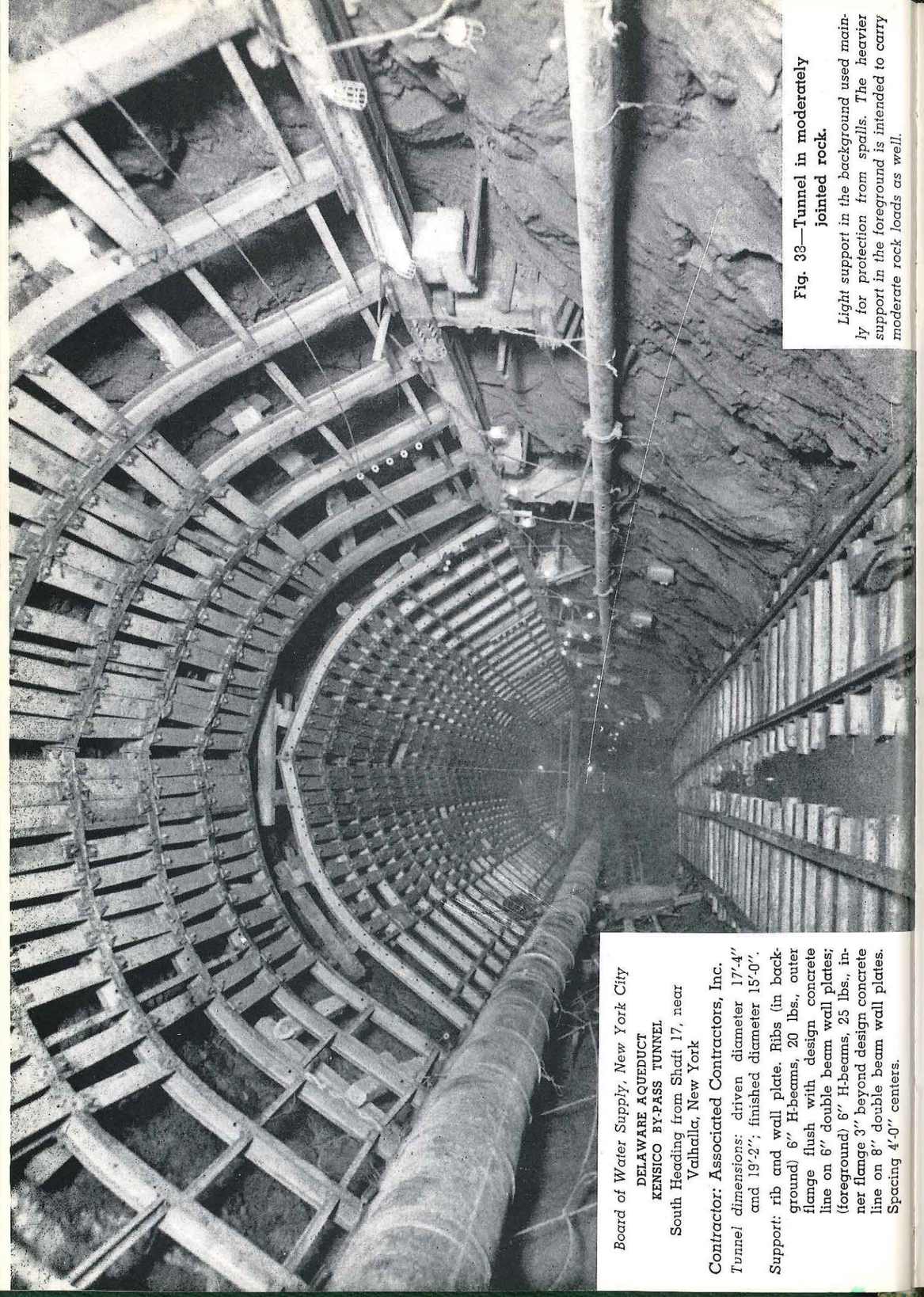


Fig. 33—Tunnel in moderately jointed rock.

Light support in the background used mainly for protection from spalls. The heavier support in the foreground is intended to carry moderate rock loads as well.

Board of Water Supply, New York City
DELAWARE AQUEDUCT
KENSICO BY-PASS TUNNEL
South Heading from Shaft 17, near Valhalla, New York

Contractor: Associated Contractors, Inc.

Tunnel dimensions: driven diameter 17'-4" and 19'-2"; finished diameter 15'-0".

Support: rib and wall plate. Ribs (in background) 6" H-beams, 20 lbs., outer flange flush with design concrete line on 6" double beam wall plates; (foreground) 6" H-beams, 25 lbs., inner flange 3" beyond design concrete line on 8" double beam wall plates. Spacing 4'-0" centers.

Some years ago the author investigated samples of what was considered by the tunnel force to be swelling rock. The tests showed that the volume of the rock was entirely unaffected by immersion. As a matter of fact, it was found afterwards that it was easy to hold the pressure with a tightly wedged tunnel support.

Specimens of decomposed gneiss which caused considerable trouble during the construction of the St. Gotthard Tunnel in Switzerland expanded after immersion by about 2.9 per cent in the direction of the cleavage.[1] The volume of a specimen of shale was even found to increase during immersion by 12 per cent,[2] whereas tests by the author on specimens of several shales from other localities, including Pennsylvania, disclosed no measurable volume change. These observations indicate that the differences between the swelling properties of different shales can even be more important than those between the swelling properties of different clays, illustrated by *Fig. 31 c*. Mere visual inspection of a shale does not furnish any information on this vital property of the material.

Occasionally important heaves and excessive pressure have been experienced when tunneling through rock formations containing layers of anhydrite. Thus for instance in a railroad tunnel in the Alps through almost horizontal strata composed of shale, gypsum and anhydrite, the tunnel floor rose during the first few years at a rate of about ten inches per year. In other tunnels through similar formations the timbering has been repeatedly crushed.

Both the heave and the pressure in such rock formations are due to the transformation of anhydrite into gypsum, which is associated with a very important increase in volume. Anhydrite is not stable in the presence of water. Hence it occurs only between practically impervious layers which keep the water away. These layers can be very thin, but they must be tight. During excavation of the tunnels the layers of anhydrite and their seals were injured and cracked; water percolated into the fissures out of adjoining water bearing strata, whereupon the anhydrite along the fissures was transformed to gypsum. The local swelling pressure caused further cracking and increased the surface of contact between water and anhydrite and this process went on until chemical alteration was complete. Hence in mining through anhydrite-bearing formations, utmost care is required to prevent injury to the layers of anhydrite and their seals.

1. F. M. Stapff, Jahrbuch der preussischen geologischen Laudesanstalt, 1888.
2. Daniel E. Moran, Letter to the Editor, Eng. News-Record, Vol. 95, December 10, 1925.

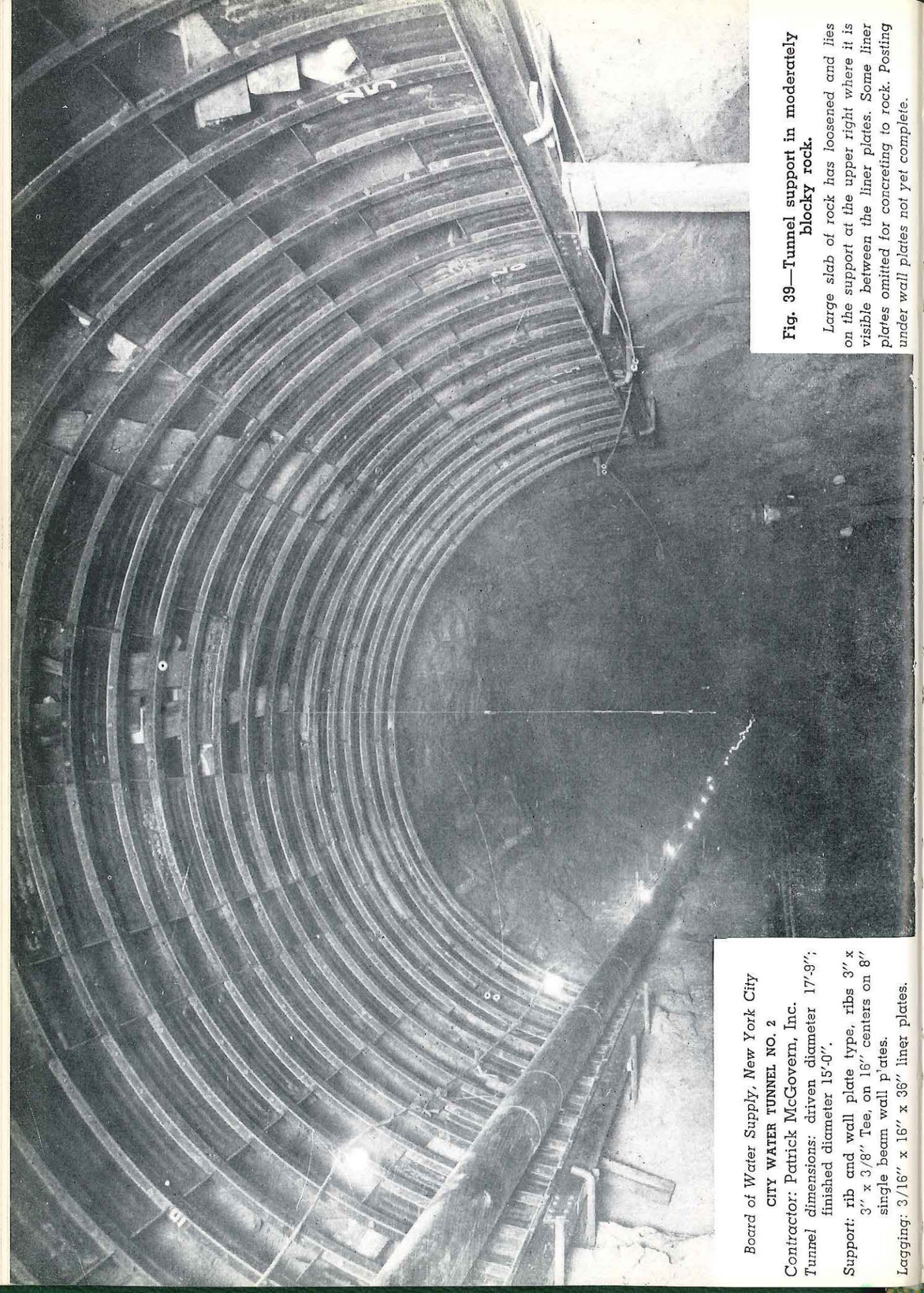


Fig. 39—Tunnel support in moderately blocky rock.

Large slab of rock has loosened and lies on the support at the upper right where it is visible between the liner plates. Some liner plates omitted for concreting to rock. Posting under wall plates not yet complete.

Board of Water Supply, New York City
CITY WATER TUNNEL NO. 2
Contractor: Patrick McGovern, Inc.
Tunnel dimensions: driven diameter 17'-9"; finished diameter 15'-0".
Support: rib and wall plate type, ribs 3" x 3" x 3/8" Tee, on 16" centers on 8" single beam wall plates.
Lagging: 3/16" x 16" x 36" liner plates.

CHAPTER 5

FORECAST OF PRESSURE AND WORKING CONDITIONS IN ROCK TUNNELS

INTRODUCTION

This chapter contains a resume of all the information which has been presented in the preceding chapters. It is intended to inform the tunnel builder on the steps required to get a conception of the pressure and working conditions which have to be anticipated in the construction of a proposed tunnel at a given site.

The review of the factors to be considered in support design is followed by a discussion of the relation of the general geology of the site and the relative importance of the inevitable uncertainties associated with the forecasts and by a description of available methods of reducing tunnel hazards to the minimum compatible with the geology of the site. The chapter will be concluded by a list of sources of useful information concerning general and regional geology.

GEOLOGICAL SURVEY

The first and foremost requirement for making a forecast of the working conditions in a proposed rock tunnel is a geological section through the center line of the tunnel, showing the approximate position of the boundaries between the different types of rocks and of all those faults or fault zones which were discovered during the survey. Since a geological survey cannot be expected to furnish conclusive information regarding many features which are vital to the tunnel builder, the conclusions should be expressed in terms of most favorable and most unfavorable possibilities.

From an engineering point of view, a knowledge of the type and intensity of the rock defects may be much more important than that of the types of rock which will be encountered. Therefore during the survey rock defects should receive special consideration. The geological report should contain a detailed description of the observed rock defects in geological terms. It should also contain a tentative classification of the defective rocks in the tunnel man's terms, such as blocky and seamy, squeezing or swelling rock. A reliable diagnosis of swelling rock can be obtained only by means of tests on samples in their natural state, before they were allowed to lose any moisture by evaporation. Since swelling conditions have a decisive influence on the speed and cost of tunneling, every shale and any other rock which may be capable of swelling should be carefully investigated and the measured volume increase should be recorded.

TERMS DESCRIBING ROCK CONDITION

On the basis of the geological report and the geological profiles a tunnel profile is prepared showing the rock conditions which are likely to prevail in the different parts of the tunnel.

In *Chapter 4,* the following terms have been used to describe the condition of rocks: Intact, stratified, moderately jointed, blocky and seamy, crushed, squeezing, and swelling.

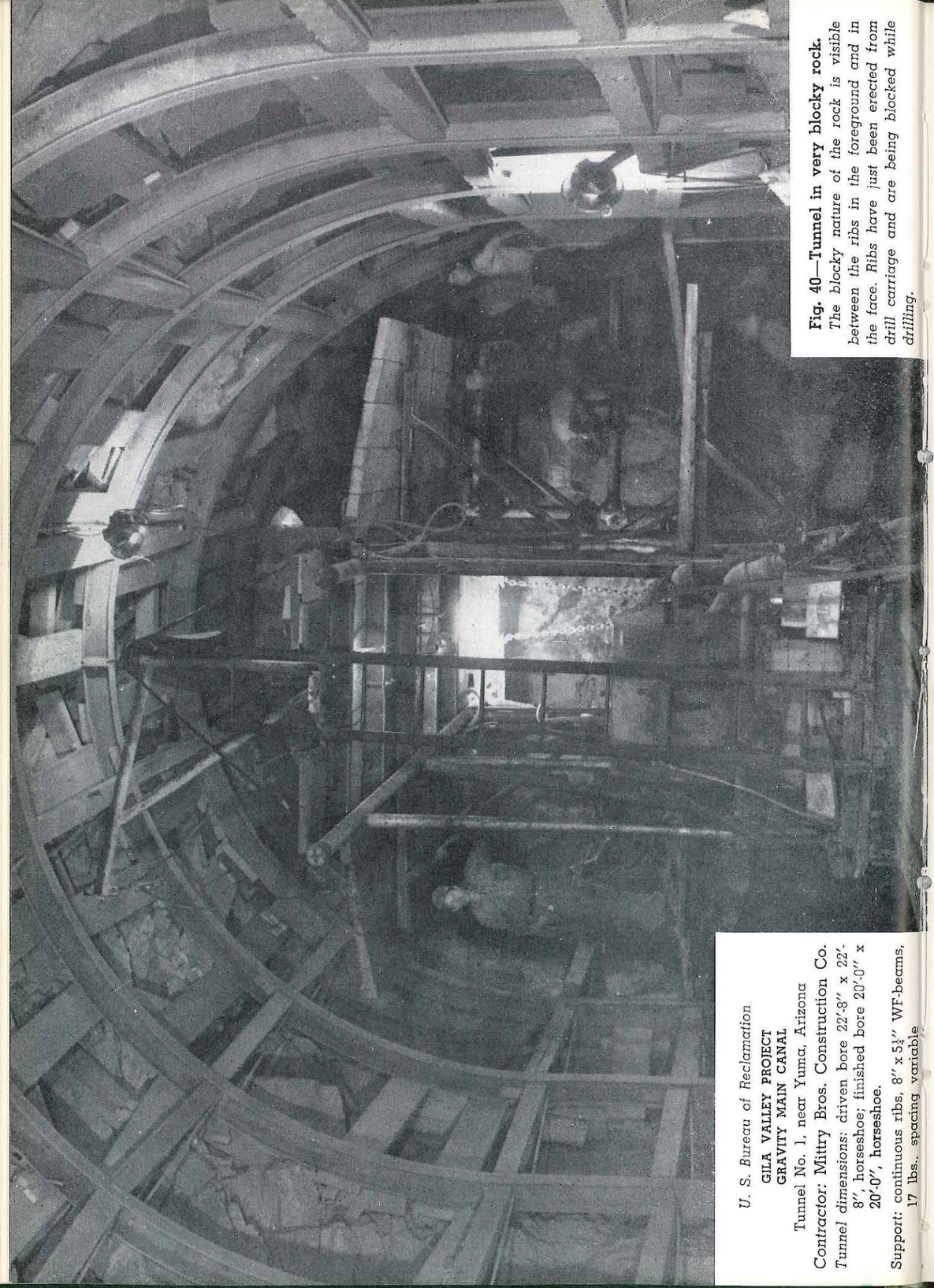


Fig. 40—Tunnel in very blocky rock.
The blocky nature of the rock is visible between the ribs in the foreground and in the face. Ribs have just been erected from drill carriage and are being blocked while drilling.

U. S. Bureau of Reclamation
GILA VALLEY PROJECT
GRAVITY MAIN CANAL
Tunnel No. 1, near Yuma, Arizona
Contractor: Mittry Bros. Construction Co.
Tunnel dimensions: driven bore 22'-8" x 22'-8", horseshoe; finished bore 20'-0" x 20'-0", horseshoe.
Support: continuous ribs, 8" x 5¼" WF-beams, 17 lbs., spacing variable

Intact rock contains neither joints nor hair cracks. Hence if it breaks it breaks across sound rock. On account of the injury to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as *spalling* condition. Hard, intact rocks may also be encountered in the *popping* condition involving the spontaneous and violent detachment of rock slabs from sides or roof.

Stratified rock consists of individual strata with little or no resistance against separation along the boundaries between strata. The strata may or may not be weakened by transverse joints. In such rock, the spalling condition is quite common.

Moderately jointed rock contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. In rocks of this type both the spalling and the popping condition may be encountered.

Blocky and seamy rock consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock vertical walls may require support.

Crushed but chemically intact rock has the character of a crusher run. If most or all of the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.

Squeezing rock slowly advances into the tunnel without perceptible volume increase. Prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or of clay minerals with a low swelling capacity.

Swelling rock advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks which contain clay minerals such as montmorillonite, with a high swelling capacity.

In practice there are no sharp boundaries between these rock categories and the properties of the rocks indicated by each one of these terms can vary between wide limits.

ESTIMATE OF ROCK LOAD

Even a very conscientious and expertly conducted geological survey of the site of a proposed tunnel cannot accomplish more than a very crude estimate of the length of the tunnel sections in which each of the principal types of rock conditions will be encountered. Further differentiation cannot be expected. Hence, even if methods for accurately computing the rock load under given rock conditions were available, they would have very little practical value on account of the inevitable uncertainties associated with predicting the rock conditions. Approximate values for the rock loads to be anticipated under the principal rock conditions are all that tunnel practice requires. The geological factors which determine the rock load were discussed in *Chapter 4*. Table 3 contains a summary of the conclusions. Since there are no well defined boundaries between the different conditions, the rock load corresponding to each rock condition is represented in the table not by a single value but by a range.

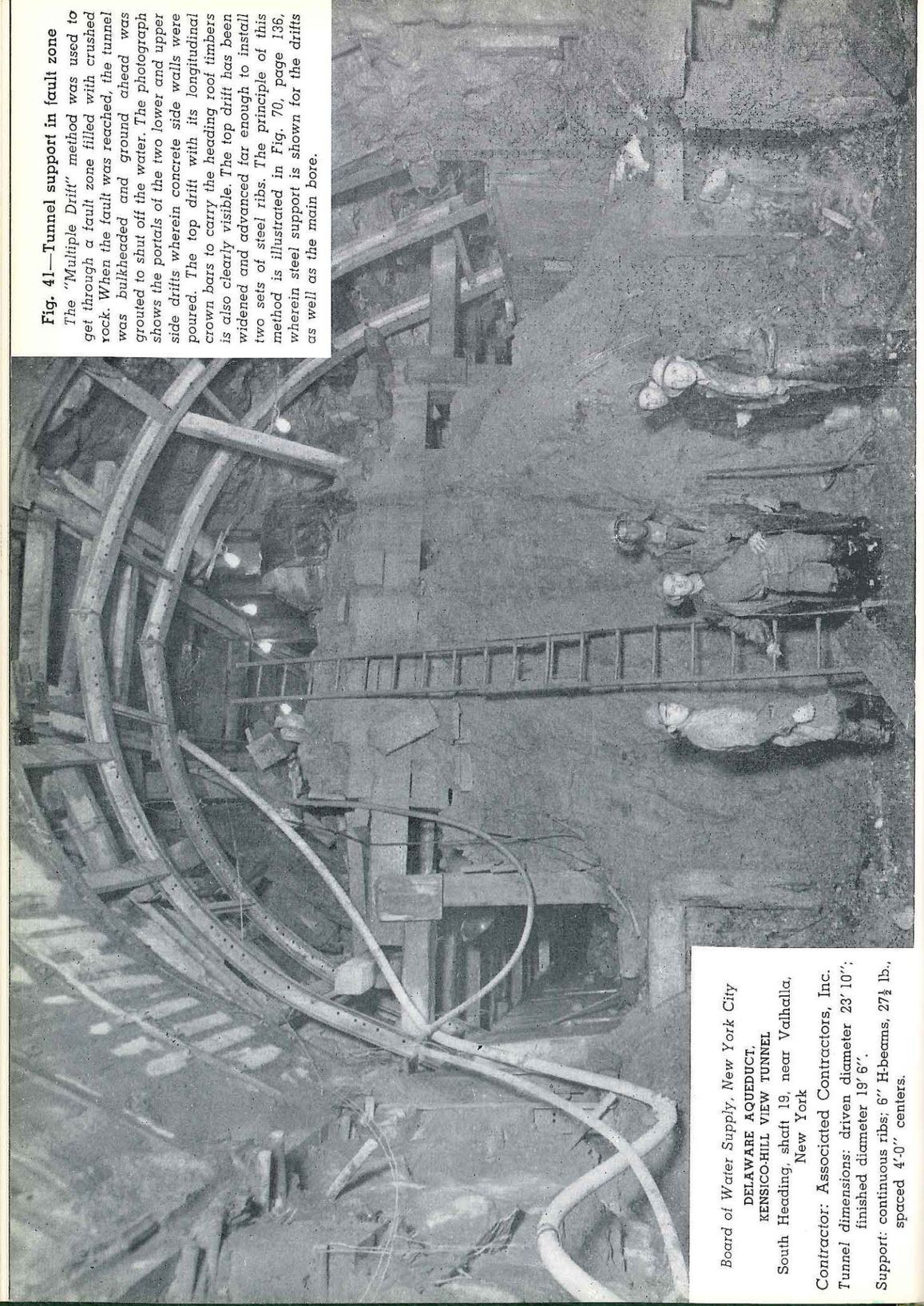


Fig. 41—Tunnel support in fault zone

The "Multiple Drift" method was used to get through a fault zone filled with crushed rock. When the fault was reached, the tunnel was bulkheaded and ground ahead was grouted to shut off the water. The photograph shows the portals of the two lower and upper side drifts wherein concrete side walls were poured. The top drift with its longitudinal crown bars to carry the heading roof timbers is also clearly visible. The top drift has been widened and advanced far enough to install two sets of steel ribs. The principle of this method is illustrated in Fig. 70, page 136, wherein steel support is shown for the drifts as well as the main bore.

Board of Water Supply, New York City
DELAWARE AQUEDUCT,
KENSICO-HILL VIEW TUNNEL
South Heading, shaft 19, near Valhalla, New York
Contractor: Associated Contractors, Inc.
Tunnel dimensions: driven diameter 23′ 10″; finished diameter 19′ 6″.
Support: continuous ribs; 6″ H-beams, 27½ lb., spaced 4′-0″ centers.

TABLE 3

Rock load H_p in feet of rock on roof of support in tunnel with width B (ft) and height H_t (ft) at depth of more than 1.5 $(B + H_t)$.[1]

Rock Condition	*Rock Load H_p in feet*	*Remarks*
1. Hard and intact	zero	Light lining, required only if spalling or popping occurs.
2. Hard stratified or schistose[2]	0 to 0.5 B	Light support. See *Fig. 38.*
3. Massive, moderately jointed	0 to 0.25 B	Load may change erratically from point to point.
4. Moderately blocky and seamy	0.25 B to 0.35 $(B+H_t)$	No side pressure. See *Fig. 39.*
5. Very blocky and seamy	(0.35 to 1.10) $(B+H_t)$	Little or no side pressure. See *Fig. 40.*
6. Completely crushed but chemically intact	1.10 $(B+H_t)$	Considerable side pressure. Softening effect of seepage towards bottom of tunnel requires either continuous support for lower ends of ribs *(Fig. 41)* or circular ribs *(Fig. 42).*
7. Squeezing rock, moderate depth	(1.10 to 2.10) $(B+H_t)$	Heavy side pressure, invert struts required. Circular ribs are recommended.
8. Squeezing rock, great depth	(2.10 to 4.50) $(B+H_t)$	
9. Swelling rock	Up to 250 ft. irrespective of value of $(B+H_t)$	Circular ribs required. In extreme cases use yielding support.

1. The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 can be reduced by fifty per cent.

2. Some of the most common rock formations contain layers of shale. In an unweathered state, real shales are no worse than other stratified rocks. However, the term shale is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so-called shale may behave in the tunnel like squeezing or even swelling rock.

   If a rock formation consists of a sequence of horizontal layers of sandstone or limestone and of immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the so-called shale and rock is likely to reduce very considerably the capacity of the rock located above the roof to bridge. Hence, in such rock formations, the roof pressure may be as heavy as in a very blocky and seamy rock.

NUMERICAL EXAMPLE OF ROCK LOAD COMPUTATION

To illustrate the effect of the rock condition on the rock load an estimate will be made of the load on the roof support in a tunnel with a width $B = 15$ ft. and $H_t = 15$ ft. at the moderate average depth of 300 ft. in granite with a unit weight $w = 165$ lbs. per cubic foot. If the region has had a complex geological history the condition of the granite may vary within wide limits over short distances. The roof load corresponding

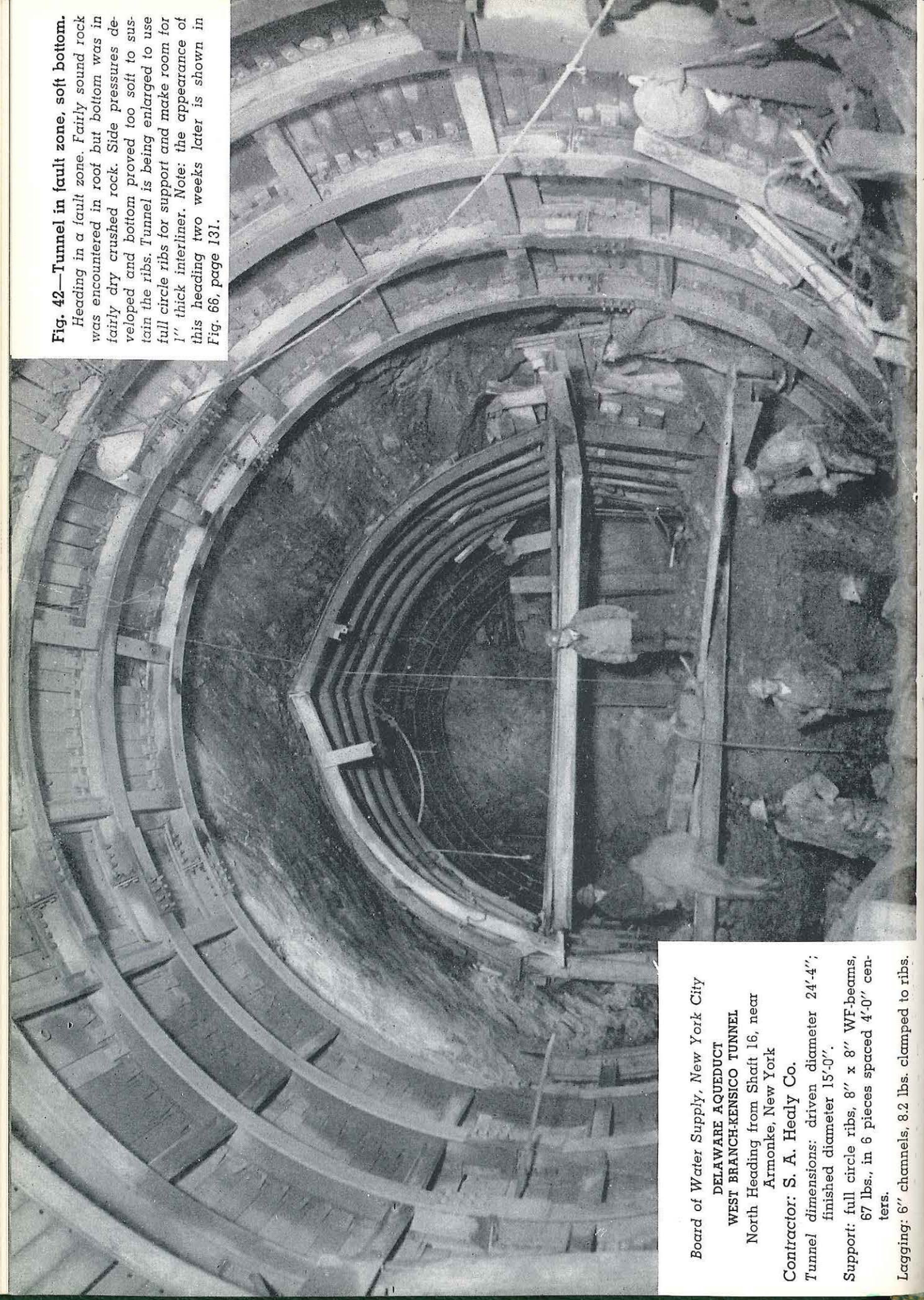


Fig. 42—Tunnel in fault zone, soft bottom.
Heading in a fault zone. Fairly sound rock was encountered in roof but bottom was in fairly dry crushed rock. Side pressures developed and bottom proved too soft to sustain the ribs. Tunnel is being enlarged to use full circle ribs for support and make room for 1" thick interliner. Note: the appearance of this heading two weeks later is shown in Fig. 66, page 131.

Board of Water Supply, New York City
DELAWARE AQUEDUCT
WEST BRANCH-KENSICO TUNNEL
North Heading from Shaft 16, near Armonke, New York
Contractor: S. A. Healy Co.
Tunnel dimensions: driven diameter 24'-4"; finished diameter 15'-0".
Support: full circle ribs, 8" x 8" WF-beams, 67 lbs., in 6 pieces spaced 4'-0" centers.
Lagging: 6" channels, 8.2 lbs. clamped to ribs.

to the various conditions which are likely to be encountered may be obtained by introducing the values **B**, $\mathbf{H_t}$, and **w** into the preceding equations. The results are as follows:

	Minimum		*Maximum*
Spalling state - - - - - - - - - -	0	to	400 lbs. per sq. ft.
Moderately jointed			
$\mathbf{H_{p\ min}}$ - - - - - - - - - - -	0		
$\mathbf{H_{p\ max}} = .25\mathbf{B} = .25 \times 15$ ft. $= 3.75$ ft $=$ - -			620 lbs. per sq. ft.
Moderately blocky and seamy			
$\mathbf{H_{p\ min}} = \mathbf{H_{p\ max}}$ for moderately jointed - -	620		lbs. per sq. ft.
$\mathbf{H_{p\ max}} = .35\ (\mathbf{B} + \mathbf{H_t}) = .35\ (15 + 15) = 10.5$ ft. $=$			1730 lbs. per sq. ft.
Intensely shattered but chemically unaltered			
$\mathbf{H_{p\ min}} = \mathbf{H_{p\ max}}$ for moderately blocky - -	1730		lbs. per sq. ft.
$\mathbf{H_{p\ max}} = 1.10\ (\mathbf{B} + \mathbf{H_t}) = 1.10\ (15 + 15) = 33.0$ ft. $=$			5440 lbs. per sq. ft.

If the granite has been chemically altered, squeezing and even swelling conditions may prevail. Loads may then, at this moderate depth, attain the following values:

	Maximum
Squeezing $\mathbf{H_p} = 2.10\ (\mathbf{B} + \mathbf{H_t}) = 2.10 \times (15 + 15) = 63$ ft. $=$	10400 lbs. per sq. ft.
Swelling - - - - - - - - - -	20000 to 40000 lbs. per sq. ft.

In the squeezing and swelling stretches full circle ribs should be used. Hence it is advisable to include an adequate supply of such ribs in the first procurement.

BRIDGE-ACTION AND LOAD-INCREASE PERIOD

The influence of the condition of the rock on the bridge-action and the load increase period is graphically represented by *Fig. 43*. *Fig. 43 a* is a vertical section through the centerline of a tunnel. In *Fig. 43 b* the abscissas represent the time (see legend to figure) and the ordinates of the dash lines the height of overbreak, if after the last round is fired, mining is discontinued and no support installed. The ordinates of the full lines represent the load on the top of the roof in feet of rock.

The bridge-action period t_b has been defined as the time which elapses between firing the round and the beginning of the breakdown of the unsupported part of the roof. In intact rock and in moderately jointed rock the bridge action period is practically unlimited and the support merely serves to protect the operations from falling rock fragments.

For blocky and seamy rock or for squeezing, slaking rock it may range between several hours and several days. Nevertheless bridge-action periods of less than several hours are quite often encountered in rock tunnels. For any given rock the period depends to a large extent on the length of the unsupported roof. Therefore it can be regulated within fairly wide limits.

When preparing the diagram *Fig. 43 b* it was assumed that the bridge-action period of all the rocks represented in the graph is equal to four-fifths of the duration of one excavating cycle. Yet, whatever the bridge-action period may be, excessive overbreak can be avoided only by installing the roof support before the period expires. Therefore the length of the period has a decisive influence on the sequence of operations in mining the tunnel.

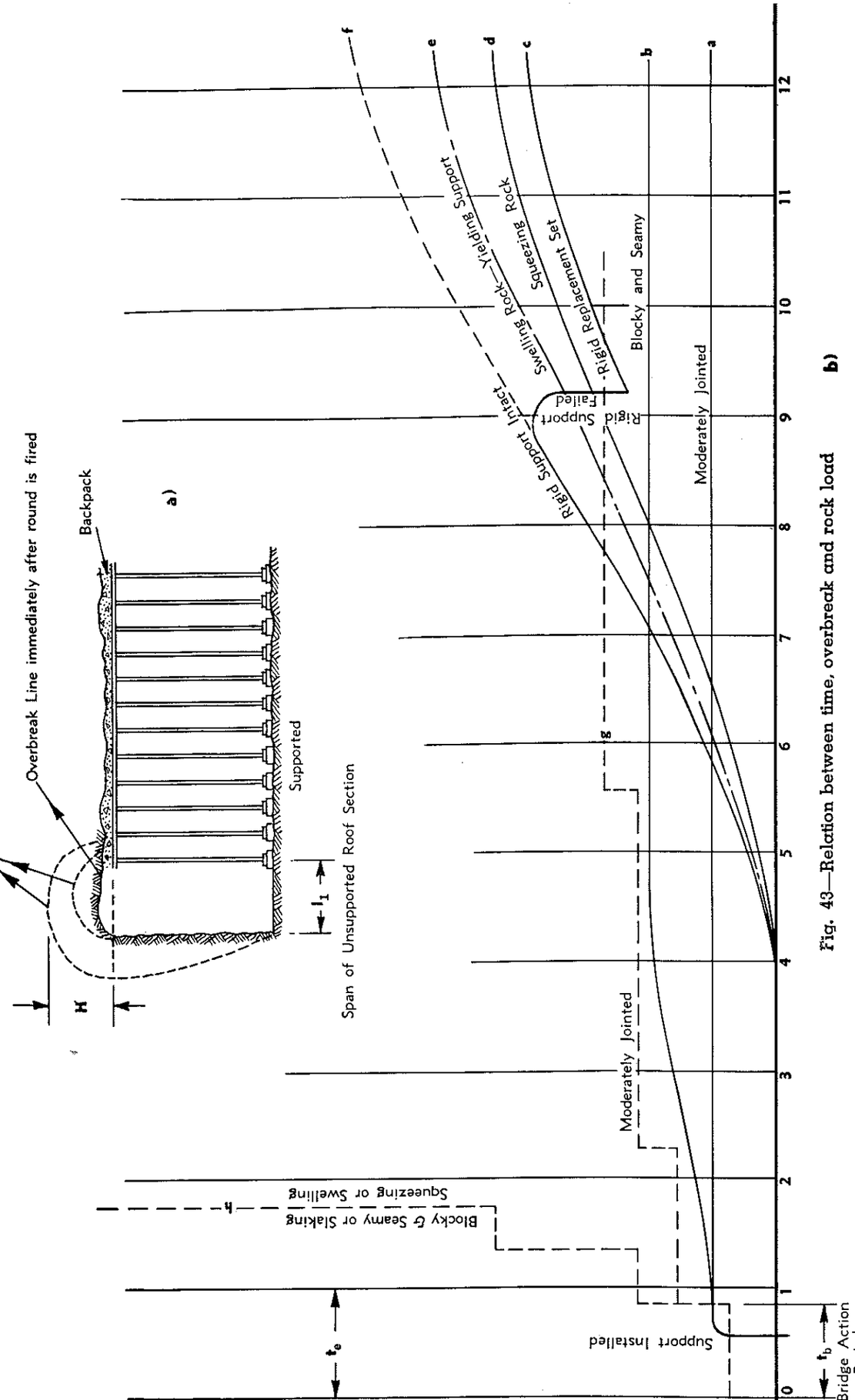


Fig. 43—Relation between time, overbreak and rock load

Abscissas of dash lines **g** and **h** represent time in excavating cycles and the ordinates the height **H** of overbreak, if excavation is discontinued and roof next to working face remains unsupported. Abscissas of full lines **a** to **f** represent time in days and the ordinates the rock load H_p in feet of rock. H_p is likely to increase less rapidly than shown in the diagram.

It is customary and economical to keep the distance between the working face and the last support as long as the condition of the rock permits. Hence if a tunnel passes abruptly and unexpectedly from a rock formation with a long bridge-action period into a formation with a short one, local but excessive overbreak is inevitable.

After the tunnel support is installed the pressure on the support may remain almost unchanged. However it may also increase to many times its initial value as shown in *Fig. 43 b*. The time which elapses until the pressure becomes fairly constant has been referred to as the load-increase period. It ranges between a few weeks and many months. The tunnel support must be designed on the basis of the heaviest rock load which it has to sustain during its service period.

The presence of large, empty spaces between support and rock is likely to increase the length of the load-increase period and the amount of load-increase very considerable. It also leads to a gradual deterioration of the rock surrounding the tunnel. Therefore the necessity of careful back-packing and wedging cannot be overemphasized. After the permanent lining is constructed the voids in the backpack should be grouted without delay.

In the early days of tunnel construction the tunnel builder was compelled to leave, in bad stretches a large amount of timber between the rock and the permanent lining. The decay of the timber had the same effect as inadequate backpacking. Furthermore backpacking itself did not always receive the serious attention which it deserves. As a consequence many tunnels developed with age various defects such as distortion of the tunnel cross-section associated with the formation of large cracks in the tunnel walls, subsidence of the ground surface above the tunnel or both combined. Every year several tunnels arrive in a state in which radical reconstruction becomes imperative. One of these is shown in *Fig. 44*.

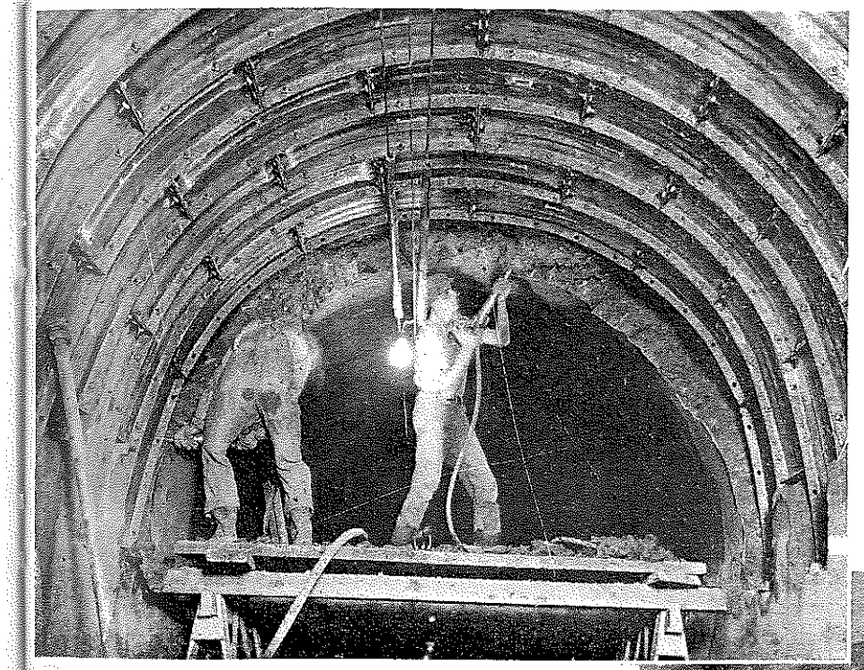


Fig. 44—Repair of damage to permanent tunnel lining in squeezing rock.

Deformed and broken concrete lining in this flow tunnel made extensive rebuilding necessary.

Old concrete was chipped out to true line where it had been squeezed into the tunnel. Liner plates were installed, covered with expanded metal, and then gunited. Space between liner plates and old concrete was grouted. Space left by decay of old wood packing outside the concrete lining was filled by

Modern tunnel practice eliminates the necessity of subsequent tunnel reconstruction by using steel supports instead of timber and by concreting against the rock or careful backpacking. Both procedures combined establish intimate contact between the rock and the permanent lining. Once such contact is established the permanent lining will always retain its shape, because the passive resistance of the adjoining rock does not permit any distortion. An exception could be in squeezing or swelling rock which sloughs locally or is overmined and surrounding rock moves into the void. This can constitute a "soft" spot and allow the support to deflect outward as heavy loads come on other areas of the support. Serious deformation and failure may result.

Clearance between tunnel support and rock is justifiable only when mining through the worst types of swelling rock. Yet even in such rocks it should only be used as a last resort, and very carefully done because only rarely does the rock move inward uniformly.

TUNNEL HAZARDS

Varieties of tunnel hazards

The term *tunnel hazard* indicates unanticipated sources of expense and delay. The hazards are due to the departure of the structural details of a given mass of rock from the statistical average for similar masses. On account of this departure the cost of a proposed tunnel may be lower, but it may also be considerably higher than the average cost of similar tunnels which have been built in similar bodies of rock.

The unanticipated difficulties may reside in abnormally high rock load, abrupt transitions from long to short bridge-action periods, excessive inflow of water and the presence of harmful gasses. The importance of the financial risks associated with these hazards depend to a large extent on the general geology of the region. The following paragraphs illustrate this interdependence.

Hazards in limestone and sandstone

Above the water table hazards are less critical than below. Below the water table sandstone is much less hazardous than limestone because in contrast to limestone it is not likely to contain underground channels or reservoirs. In limestone, heavy, concentrated flows of water or of a mixture of sand and water may be encountered. In zones consisting of crushed sandstones or limestone, earth tunneling methods must be used, but if such zones are present at all they are commonly very narrow. Wherever the rock is not crushed, the rock load nowhere exceeds moderate values. The occurence of harmful gases such as CO_2 or H_2S is uncommon except in regions of post-volcanic activities.

Hazards in formations containing shale

Tunneling conditions in rock formations containing shale depend chiefly on the character of the shale. This character may range anywhere between that of a sound rock and of a stiff, swelling clay. However, within any one shale formation, the tunneling conditions are likely to be fairly uniform. The rock load is either consistently low or consistently high. The quantity of water which enters the tunnel from shale is commonly low, but large flows may be encountered wherever a water-bearing formation rests on shale.

Shales are sometimes associated with layers of coal, or anhydrite. In coal bearing shale foundations, the explosive gas, methane (CH_4), has been encountered repeatedly. If tunneling operations injure the seal of layers of anhydrite the water which percolates through the cracks in the anhydrite gradually changes the anhydrite into gypsum and heavy swelling pressure ensues. The water which flows across anhydrite strata into the tunnel contains calcium sulphate which attacks concrete. It is also likely to contain hydrogen sulfide (H_2S) which is lethal even in moderate concentration.

Hazards in schists

In unaltered schists the rock load ranges between zero and moderate. Although the inevitable overbreak and the inflow of water may be quite important, the hazards associated with unaltered schists are rather mild. However, in schists chemical alterations are very common. In decomposed schists squeezing and even swelling conditions may be encountered. These conditions are associated with very high or excessive rock load. In many railroad tunnels through altered schists heavy loads were combined with large discharge of water into the tunnel.

Hazards in intrusive igneous rocks

In intrusive igneous rocks such as granite many tunnels with great length have been built without requiring any support and without interference by water. Yet, from time to time, extremely unfavorable conditions have been encountered, involving heavy flow of water in some parts of the tunnel and squeezing or swelling conditions in others. General experience with tunneling through these rocks indicates that the probability of hazards is very low, but the deviations from normal can be important enough to upset completely the original estimate of costs. Furthermore, in many instances the existence of abnormal conditions at depth cannot be predicted from surface evidence. Hence the intrusive rocks should be considered decidedly treacherous.

Hazards in extrusive igneous rocks

Extrusive igneous rocks such as rhyolite are commonly associated with strata of volcanic tuff or breccias. The igneous rocks, the tuff and the breccias may be encountered in an advanced state of decomposition. The tuff and breccias may even be still in an unconsolidated state; large quantities of water may enter the tunnel from fault zones and in young volcanic rocks, the occurrence of harmful gases is not uncommon. Therefore tunnels through rocks of this category can be expected from the very start to be extremely hazardous. The experience on the Mono Craters Tunnel in California and the Tanna Tunnel near Atami in Japan can be considered representative for the difficulties which may be encountered in such rock formations.

Provisions for coping with tunnel hazards

A competent and experienced geologist is able to predict which types of difficulties may be encountered in different parts of a proposed tunnel, but he cannot make in advance of construction a quantitative evaluation of the difficulties. Hence the first estimate of the material and equipment required for constructing a tunnel inevitably involves a certain amount of guesswork. These uncertainties also characterize the preliminary design of the tunnel support and should be considered when preparing the initial procurement of steel support.

The cost of emergency supports such as circular steel ribs or extra heavy rib sets is small compared to the cost of meeting an emergency with inadequate supplies. Hence if the geologist indicates that unfavorable rock conditions of a certain type may be encountered, enough supplies should be kept on the job to cope with the anticipated difficulties until supplementary material arrives at the job.

If unfavorable rock conditions are anticipated, it is also advisable to supplement the standard geological survey by various other investigations intended to furnish more specific information on the rock conditions. The principal methods are diamond borings, geophysical surveys, observations in pilot tunnels and geological observations during construction.

Diamond core borings

On several important projects involving the construction of tunnels, diamond core borings have been made in advance of construction to verify or modify the conclusions derived from the geological survey. Such an investigation may be particularly profitable, if conditions permit a choice between different locations of a tunnel in a region in which difficult tunneling conditions are anticipated. In such regions the cost of the borings is usually very small compared to the savings which might result from selecting the most favorable location. In locating the tunnels for the Catskill and several other important aqueducts this method has been used extensively with noteworthy success. (See article on *Faulting and Thrusting in Relation to Tunnelling*, pp. 35 to 38).

Geophysical survey

During the last few decades methods have been developed which, under favorable conditions, permit an approximate determination of the location of the boundary at depth between rocks with very different physical properties, such as granite and shale. These methods consist in observing the effect of the sub-surface discontinuity on the intensity and geometry of a natural or an artificially created field of force or on the rate of wave propagation. These methods are known as *geophysical methods.*[1] They constitute nowadays one of the most important methods of prospecting for oil and ores. Quite recently two of these methods, the seismic and the electrical prospecting method, have also been adapted to subsoil exploration in connection with foundation and tunnel problems. The success of these methods depends chiefly on the importance of the difference between the physical properties of the rocks located on either side of the unknown boundary.

The electrical resistivity, and the modulus of elasticity which determines the rate of wave propagation of a crushed or completely decomposed rock, are very much lower than the corresponding value for the same rock in a relatively intact state. Therefore the methods have been successfully used for determining the boundary between sound and defective rocks on tunnel jobs. Since geophysical rock exploration is very much cheaper than the boring method, it deserves at least a trial. If the attempt is successful, the number of core borings required to secure a given amount of information can be substantially reduced.

Function of pilot tunnels

The pilot tunnel serves the purpose of a large-diameter exploratory hole. It also drains the rock ahead of the main excavation. If the inflow of water is excessive, the rock can be grouted before the main excavation reaches the water-bearing zone. Heavy squeezing or swelling pressures are detected in time to order the required tunnel supports for the main excavation and to revise the design for the permanent tunnel support. It even permits starting the main excavation at several points simultaneously in order to reduce the time of construction. Hence, if a tunnel with a large cross section is located in an intensely folded or faulted region, the construction of a pilot tunnel should not be omitted.

Geological observations during construction

Whatever the method of attack in a tunnel may be, it is always advantageous to know the details of the character of the rock as far ahead of the working face as conditions permit. However, the extrapolation from the exposures in the tunnel to the

1. See for instance C. A. Heiland, Geophysical Exploration, Prentice-Hall Inc., New York, 1940.

rock conditions beyond the working face requires geological knowledge and training. Therefore, it is always advisable to keep an experienced geologist permanently on the job. His original conceptions regarding the rock conditions are inevitably more or less inaccurate. However, his observations in the tunnel provide him with the means for correcting his original conceptions step for step whereby his capacity for correct extrapolation increases. The savings resulting from a few accurate forecasts of pending changes in the rock conditions are likely to be more important than the cost of retaining the services of a resident geologist during the entire period of construction.

SOURCES OF USEFUL INFORMATION

On account of the decisive influence of geological factors on the difficulties and costs of tunnel construction, every engineer engaged in tunnel work should be familiar with at least the elements of physical geology. The following elementary textbooks are recommended:

Arthur Holmes, "Principles of Physical Geology," New York, 1945, The Ronald Press Company. This book also contains an adequate bibliography for those readers who wish to get more advanced information on various phases of the subject.

C. R. Longwell, A. Knopf and R. F. Flint, "Outlines of Physical Geology," 2nd edition, John Wiley and Sons, Inc., New York, 1941.

Marland P. Billings, "Structural Geology," Prentice-Hall, New York, 1942.

R. F. Leggett, "Geology and Engineering," McGraw-Hill Book Co., Inc., New York, 1939, contains a chapter on tunnels.

F. H. Lahee, "Field Geology," 4th Edition, McGraw-Hill Book Co., Inc., New York, 1941.

C. F. Tolman, "Ground Water," McGraw-Hill Co., Inc., New York, 1937.

Even a moderate amount of knowledge of geology enables the tunnel engineer to take advantage of what is known about the geology of the region in which a tunnel is to be built; to decide whether or not the geological conditions at a new site require more detailed investigations and to interpret geological reports in engineering terms.

The most prolific source of preliminary information on the geology of individual parts of the United States of America are the publications of the U. S. Geological Survey. Geologic maps and brief descriptions of a few regions have been published in the Folios of the Survey. A vast amount of useful geological data has been published by the same organization in the Water Supply Papers. Papers pertaining to the geology of particular regions can be located by means of "Geologic Literature on North America," bibliographic bulletins of the U. S. Geological Survey, published every two years. Cumulative bibliographies are available for the periods 1785 to 1918 and 1919 to 1928.

SECTION II

SELECTION AND DESIGN

OF

SUPPORT

CHAPTER 6

FACTORS AFFECTING LAYOUT OF TUNNEL SUPPORT

TUNNEL CROSS-SECTION AND TUNNEL SUPPORT

Cross-section of tunnels in intact rock

The term *Rock* may bring to the lay mind the thought of solidity; of something firm and enduring. On this basis it might seem that tunnels could be driven through rock with out supporting the exposed surfaces, and that the tunnel bore would remain permanently as excavated. This would be true if tunnels were driven through intact rock, or rock having no defects. Some few tunnels are driven under these fortunate circumstances, and stretches of other tunnels may be located in intact rock.

Under these favorable conditions no support will be required. Whether the tunnel will be permanently lined will depend on the purpose of the tunnel. The rock cannot be blasted or trimmed to present a smooth surface. The rough and irregular surface generally is not objectionable in a railway tunnel or a mine drainage tunnel, but water and sewer tunnels are frequently lined to eliminate turbulence thus increasing their discharge capacity. Many highway tunnels are lined simply for good appearance.

Cross-section of tunnels in defective rock

Very few tunnels are located in perfectly intact rock throughout their whole length, the vast majority being driven through rock with defects of one kind or another, as reviewed in the preceding *Section I* on *Engineering Geology*. In general, defective rock requires support until the permanent lining can be placed.

Definition and function of tunnel support

Support, in tunnel work, may be defined as that structure which is erected in the tunnel bore promptly after blasting. It serves to protect the operation from falling rocks and to prevent invasion of the excavated space by surrounding ground while the permanent lining is being constructed.

As long as wood was the only structural material available for support, its function was usually considered as temporary only although in many earlier tunnels it served also as the permanent lining. When a permanent concrete lining was placed, the timbers had served their purpose. With the introduction of steel, this concept of support is changing. Steel embedded in concrete is as permanent as the concrete.

Service period of tunnel support

Contrary to practice in earth tunnels, a rock tunnel will ordinarily be "holed through" before the permanent concrete lining is placed. This is commonly the most economical procedure, as it avoids interference between the excavating and concreting operations, which interference adds to the expense of both. There may be some exceptions to this procedure, especially where exceedingly heavy loads are encountered in short stretches which makes it desirable to concrete them at once to hold the ground.

This practice requires the support system to sustain the load for a long period of time. Largely depending on the length of the tunnel, the support must carry the loads that build up over periods ranging from several months to several years before the permanent lining is installed. Quite frequently the time required for the rock to adjust

Fig. 45—Steel rib support replaced decayed timber support.

The service period of the wood support initially installed was so long that the wood lagging rotted out, causing a collapse. When remined, steel support was installed.

City of Los Angeles, Dept. of Water & Power
MONO BASIN PROJECT, MONO CRATERS TUNNEL
West Portal Heading, near Leevining, California

Contractor: Force Account

Tunnel dimensions: driven bore 11′-9″ wide by 12′-3½″ high, horseshoe; finished diameter 9′-7½″.

Support: continuous ribs, 6″ H-beams, 20 lbs.; normal spacing 5′-0″ but frequently spaced much closer as shown in *Fig. 108*. Invert struts were used extensively in both squeezing rock and earth sections.

itself to the new distribution of pressures is shorter than the service period. Hence, in many cases the support structure must sustain the same loads as the permanent lining.

An exceptionally long service period was required of the support in the Mono Craters Tunnel, 59812 ft. long, built by the City of Los Angeles, (Bureau of Water and Power.) This tunnel was started in 1934. The west heading, after advancing through rhyolitic tuff and granite passed through a stretch of morainal deposit. Permanent concrete was not placed in this heading until 1939. In the meantime some of the wood lagging in the morainal stretch rotted out, causing a collapse of the support and a temporary blocking of that heading. The wood support was replaced with steel ribs and wood lagging similar to that shown in *Fig. 45*.

Steel support as part of the designed structure

If a support system must be strong enough to carry the ultimate load, it appears logical to design it in such a manner that it can be considered as an integral part of the permanent lining. The use of this method is limited to steel supports, because wood supports are subject to decay, regardless of whether they are located within or outside the concrete. As a matter of fact, in many instances the decay of wood which was left behind the permanent lining has led to undesirable surface settlement. It even

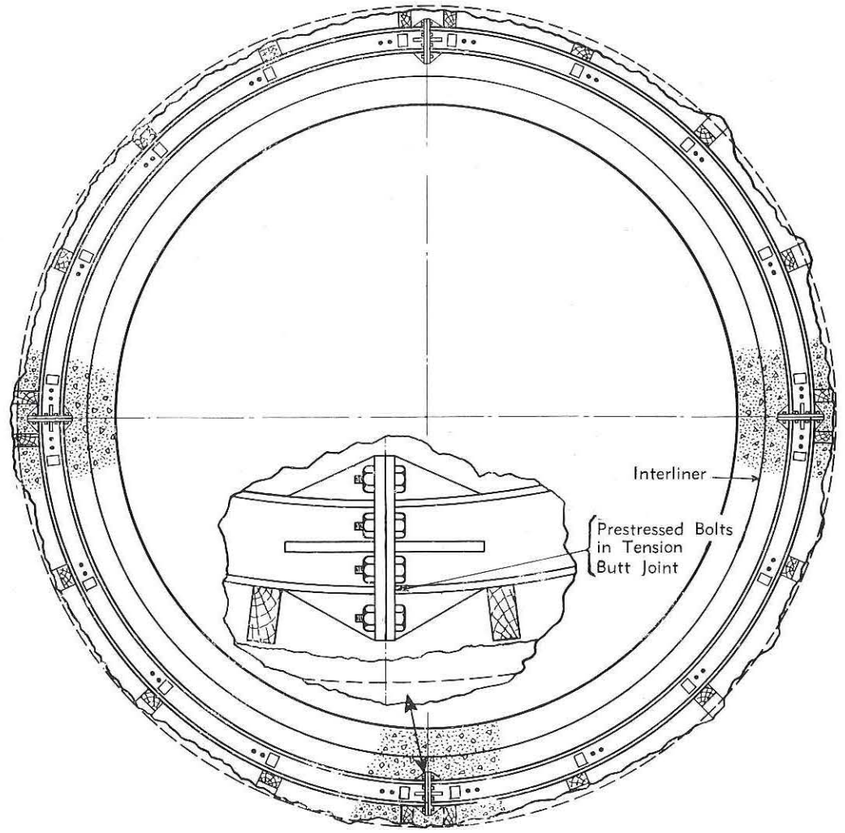


Fig. 46—Steel support used for ring tension in the finished pressure tunnel.

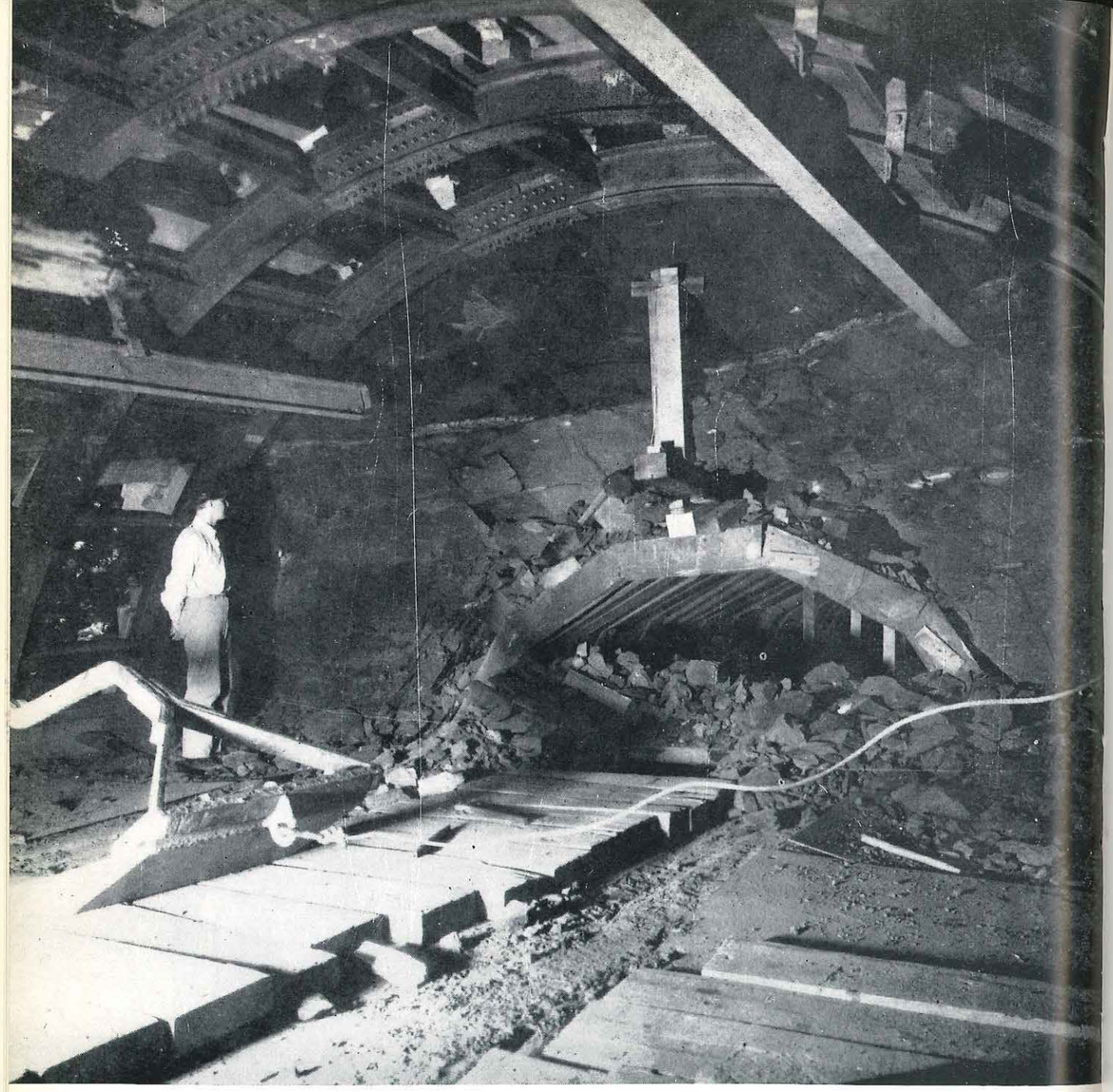


Fig. 47—Steel ribs used for two purposes.

The full circle ribs were designed to carry the rock loads during construction. Full section spliced joints enabled them to function as ring tension members in the completed structure.

The four pressure tunnels on this project were driven through shale with a center drift followed by enlarging operations by either "Full Face" or "Heading and Bench" method. The arch of the center drift is clearly shown in the background. Timbers in the left foreground may be removed to permit muck to be scraped into cars below by power operated scraper in the left foreground.

U. S. Engineer Photo

FORT PECK DAM, DIVERSION TUNNEL Montana

Contractor: Silas Mason Co., Inc., and Walsh Construction Co. *also* Force Account

Tunnel dimensions: driven diameter 32′-2″; finished diameter 24′-8″. Power Tunnel No. 1 was interlined with steel pipe.

Support: Full circle ribs, 10″ I-beams, 29 lbs., spaced at 3′-6″. Full section spliced joints.

Spreaders: 5″ I-beams, 10 lbs., 3′-6″ spacing.

Lagging: when used, 18 ga. sheets laid over spreaders as protection from spalls, and removed prior to concreting.

has caused serious damage to the permanent lining because it eliminated (over large areas of the extrados) the passive resistance whereupon the concrete lining was pressed or squeezed out of shape. See *Fig. 44*, page 95.

If a steel support is incorporated into the design concrete, the concrete serves to preserve the steel and to increase the factor of safety to the value required for permanent structures.

It is hardly ever economical to reduce the thickness of the concrete lining of a tunnel by reinforcing it with steel bars which are specifically provided for this purpose. Even the benefit derived from incorporating steel support into the permanent lining does not reside only in the strengthening effect of the ribs. The economic benefits derived from designing the steel support as a part of the permanent lining are chiefly due to a considerable reduction of the yardage of excavation and concrete thereby made possible.

Still more conspicuous are the benefits associated with incorporating the steel into the concrete of a pressure tunnel of circular cross-section, as in *Fig. 46*. The circular steel ribs are installed originally for support whereas after the tunnel is in operation they carry part or all of the tension set up by internal water pressure. Such a dual function was assigned to the ribs in the Fort Peck Dam Tunnels, *Fig. 47*. Tension butt joints were used on support for pressure tunnels at the Mangla Project in West Pakistan. Tension joints including both butt and flange splice plates were used on support for tunnels at Fort Randall, Garrison, Oahe, and numerous other reservoir projects.

Inside cross-section

The first step in designing a tunnel is to choose the inside cross-section to best serve the purpose for which the tunnel is to be built.

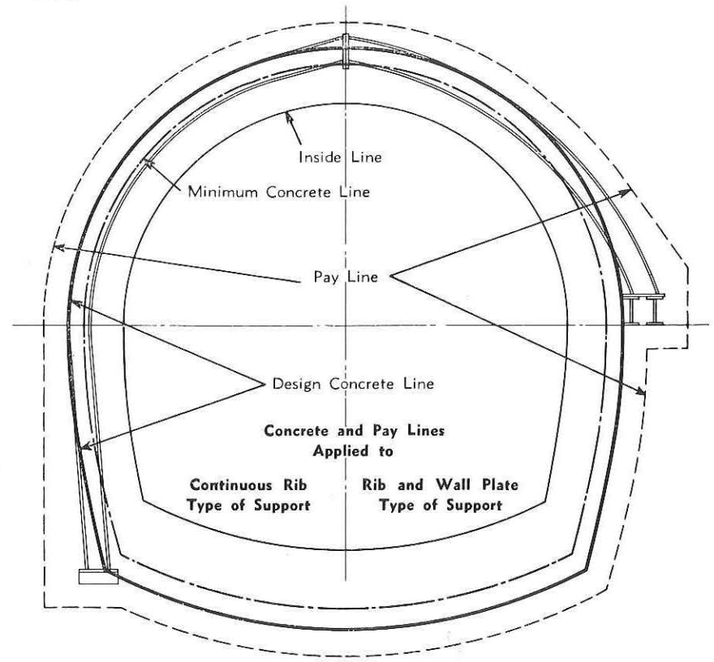


Fig. 48—Tunnel cross-sections

In intact rock the inside cross section is determined exclusively by the purpose of the tunnel, because the rock supports itself. Thus in self-supporting rock the cross-section of the tunnel can be given any desired shape. In defective rock, the choice of the cross-section may be influenced to some extent by the nature and gravity of the prevailing rock defects. The more unstable the rock, the more it is desirable to give to the tunnel section a shape approaching that of a full circle. However, the instances in which a compromise is required between the most desirable and the most practicable tunnel section are rather rare.

If two or three different inside cross-sections for a tunnel serve their purpose equally well, the decision is to be based mainly on an estimate of the costs.

Minimum concrete thickness

If the geological survey of the tunnel route indicates a possibility that part of the tunnel will be located in intact rock, the engineer has to decide on a minimum thickness of concrete for this section. He will show the *minimum concrete line* on his drawing of the tunnel cross-section and provide in his specifications that no rock may encroach within this line. See *Fig. 48.*

As intact rock imposes no load on the lining, the thickness of the lining is determined by other considerations, such as: facility of placing the concrete, depth of appurtenances that must be incorporated in the lining, cable duct sizes, etc.

Designed concrete thickness

Generally the geological survey and local experience will indicate that defective rock will be encountered somewhere along the line. Defective rock means pressure. It also means that support will be required.

The engineer must of necessity make an estimate of the magnitude this pressure will attain after the rock has adjusted itself to its new pressure conditions. Once the ultimate magnitude of the load has been estimated, the thickness of plain concrete is established to satisfy the loading assumption and a *design concrete line* is shown on the cross section, as in *Fig. 48.*

Use of steel bars and all or part of the main members of the steel support system as reinforcing may decrease the thickness of design concrete. As already noted, however, the use of reinforcing bars for the sole purpose of reducing concrete thickness in a tunnel is usually not economical. As a rule it is cheaper to put in more concrete, so that bars are not needed. However, if the rock must be supported, and if steel support is used, the steel can in some cases be utilized to reduce the concrete thickness.

In the specifications or on the plans the engineer will provide that no wood of any kind nor any lagging, either wood or steel, shall encroach on the design concrete line. He will thus insure full concrete section where support is required; however, in supported stretches, he may permit sound rock to extend to the minimum concrete line between ribs rather than require it to be slabbed off.

Overbreak and pay lines

Rock cannot be blasted to a smooth line. Yet the specifications require that no rock should project inward past the minimum concrete line and very little past the design concrete line. Hence if a smooth line is traced through the uneven boundary of the excavated space this line must be located well beyond the design concrete line. It

will be referred to as the *overbreak line.* The position of this line can only be estimated in advance of construction.

The estimated overbreak line represents the basis for establishing the *pay line* representing the outer boundary of the excavation and concrete for which payment will be made. Payment is made for the quantities enclosed by the pay line regardless of whether the contractor overshoots or undershoots. The pay line for policy reasons, may be established somewhere between the overbreak and design lines.

Regarding the location of the pay line, two procedures are used:

1. Set the pay line at some arbitrary distance from and parallel to the design concrete line. This leaves it up to the contractor as what overshooting he will do at his own expense to provide space for his support, whether of wood or steel, and also to provide width at the working floor level to accommodate his equipment.

Such a set up has been known to lead to numerous controversies in the administration of the contract, as it throws all hazards on the contractor. The resident engineer's hands are tied, as he cannot order the contractor to provide space for more adequate support should it become necessary.

2. Set the pay line at some specified distance from the outside of the steel (or wood) support system, and provide for space at the working floor line. To keep control of the pay line, this procedure requires that several alternative support systems be chosen, approved and designed in advance of construction.

During construction, the resident engineer is in position to approve or order whatever changes in the support system may be called for by an unexpected change in the rock conditions.

This latter method of placing the pay line is becoming generally adopted as it reduces the hazard to the contractor. Hence it lowers unit prices, and thus lowers the cost to the owner in case favorable conditions are encountered. It insures a safer job and better workmanship.

Pay line offset

There is considerable variation in practice in setting the distance between the support or the design concrete line and the pay line. Pay lines have been set from 5 inches to 13 inches beyond the design line or the support line. If too tight a pay line is set, unit prices for excavation and concrete will be higher and extras paid for at these prices will be unduly high. If the pay line is too liberal, unnecessarily large quantities of excavation and concrete may involve higher expense than necessary.

Contour of pay Excavation

When the pay line is established with reference to the support, it is not parallel to the design line at all points because allowance must be made for wall plate hitches, extra width at working floor level or subgrade, extra height at the crown for concrete delivery pipe clearance, etc. The amount of departure depends on the method of attack and the type of support. Hence, if the pay line is established with reference to the support, it is necessary to establish a pay line for every one of the different types of support which may appear in the contract drawings.

Because of this variation of pay lines it has become common practice among engineers to show alternate cross-sections of the tunnel, which depict several accept-

Pennsylvania Turnpike Commission
PENNSYLVANIA TURNPIKE
SIDELING HILL TUNNEL
East Portal, near Fort Littleton, Pennsylvania
Contractor: The Arundel Corporation.
Tunnel dimensions: driven bore 31'-6" wide by 25'-5" high, straight side; finished bore 28'-6" wide by 20'-8" high.
Support: rib and post, ribs 8" x 8" WF-beams, 31 lbs., posts 8" x 6½" WF-beams, 24 lbs.; spacing varied but generally was 4 to 5 ft.

Fig. 49—Reduction of size of driven bore when the support is changed from wood to steel.

A considerable reduction in quantities of both excavation and concrete usually results from use of steel support in place of timber sets. The extrados of the steel sets is usually at the design concrete line whereas the intrados of the timber support is usually at the design line.

able types of support systems. These types will be selected to match the various methods of attack that may have to be used to get through the various kinds of rock to be encountered. A pay line will be set for each type. The ultimate cross-sections are not necessarily limited to those shown on the contract drawings, but these serve as a guide for establishing others if the necessity arises during construction. The advantage of showing alternate acceptable cross-sections on the contract drawings is that the contractor gets a fairly definite idea of what to bid on which enables him to figure much closer and submit lower unit prices.

Furthermore, when it comes to getting the job started, everybody's attention is directed to installation of equipment, opening up portals or sinking shafts, etc., and consideration of the support is relegated to a later date. If support is shown, the drawings form the basis for procurement of at least the initial supply of support steel in time for the start of the job, with a minimum of encroachment on the time of busy persons. Changes, if desirable, can be made on subsequent releases.

CONSTRUCTION MATERIAL FOR TUNNEL SUPPORT

Steel versus wood support

After the engineer has established the preferred inside cross-section of the tunnel and chosen the general procedure for establishing a pay line, he should decide which material, steel or wood, will be shown as support for his basic sections. If the tunnel is long enough and of such character that a boring machine can profitably be used he may want to specify or permit an option of circular shape, in which case curved steel supports would be mandatory.

Since many tunnels are not long enough to justify use of a boring machine, conventional procedures of drilling and blasting may be used and the following discussion comparing steel and timber supports relates more particularly to this method of operation.

Present trends

Within the second quarter of this century steel has become almost universally adopted as the standard material for support in rock tunnels. Before that, timber sets were used almost exclusively.

The trend from wood towards steel is due partly to the fact that the use of steel for tunnel supports reduces the cost of the tunnel, partly to the fact that it reduces very considerably the number of skilled workmen required to erect the support, and partly to the fact that steel encased in concrete is permanent. Part of the saving in cost is due to an increase in the speed of tunnel driving, and another part is due to a reduction of the yardage of excavation and concrete for a given inner cross-section of the finished tunnel. The following paragraphs deal with the factors which need to be considered when choosing between wood and steel. The discussion will be concluded by an example of steel-wood comparison.

Speed and economy

Development of pneumatic rock drills, the advent of the drill carriage, or jumbo and perfecting of mucking equipment tremendously increased the speed of tunnel driving. They cut hours off the excavating cycle. Steel supports, that could be erected in minutes instead of hours, cut more time off the cycle. Futhermore, steel support reduced the cross section of the tunnel, *(Figs. 49 and 50)* thereby reducing drilling and mucking time still further.

The result was a gain of one, two, or even three rounds in a 24-hour day. This extra advance per day greatly lowered the cost per foot of tunnel, as it spread the monthly overhead charges over a much greater footage. Even greater speed of advance can be achieved with a boring machine.

Fig. 50—Change from wood to steel support reduces size of driven bore

Important savings in excavation and concrete result from the smaller driven bore made possible by steel support.

Metropolitan Water District of Southern California
LOS ANGELES-COLORADO RIVER AQUEDUCT, WEST EAGLE MOUNTAIN TUNNEL
near Mecca, California

Contractor: Broderick and Gordon
Tunnel dimensions: driven bore, steel, 18'-2" wide by 17'-10" high; timber 19'0" wide by 18'-8" high; finished bore, 16'-0" by 16'-0" horseshoe.
Support: continuous ribs, 6" I-beams, 12.5 lbs., spacing variable.

Cost of erection

Steel sets can be erected in a fraction of the time it takes to erect a timber set, and by fewer men. This means not only a big direct saving in erecting labor cost itself, but also a big reduction in stand-by time of other workmen waiting to start their work.

Handling of support members

An intangible saving for steel is the labor cost of handling above ground and underground. The few pieces in a steel set are supplied to the heading with less labor than is required for the numerous bulky pieces of a timber set.

Storage space requirements

In some cases space at the top of the shaft is at a premium. Steel ribs store in considerably less space than timber sets.

Skill required for support erection

The preparation of the mitered joints of timber sets and the blocking of the timber sets in the tunnel requires skill and experience on the part of each carpenter engaged in the work. No special skill is required to erect steel sets although a certain amount of judgment is required in blocking. Even blocking requires less skill with steel sets, as the curved rib permits blocking wherever a favorable point in the overbreak is found, providing the maximum permissible block spacing is not exceeded. The timber sets require blocking at the mitered joints, whether overbreak conditions at these points are good or not.

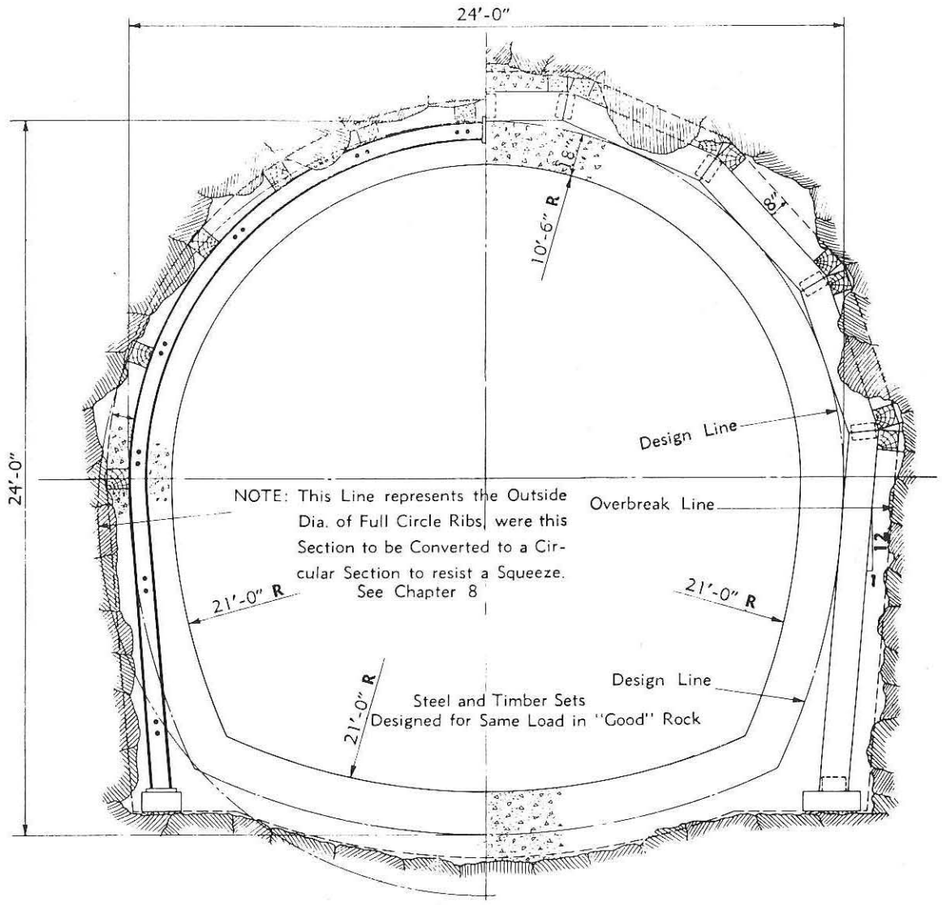

24'-0"
24'-0"
10'-6" R
38°
8"
Design Line
NOTE: This Line represents the Outside Dia. of Full Circle Ribs, were this Section to be Converted to a Circular Section to resist a Squeeze. See Chapter 8
Overbreak Line
12"
21'-0" R
21'-0" R
21'-0" R
Design Line
Steel and Timber Sets Designed for Same Load in "Good" Rock

Fig. 51—Comparison of steel and wood supports

Quantities of excavation and concrete

The fact that steel ribs can be safely embedded in the design concrete reduces the cross-section of the bore, thereby making important reductions in the quantity of both excavation and concrete.

For instance, a tunnel requiring an 8-in. steel rib to support the roof would require a 12-in. or 16-in. timber set. This timber would require 12 in. or 16 in. of additional excavation all around the periphery of the tunnel, plus corners, above the bottom of the support. Furthermore, that additional space must be filled with concrete which has no other purpose than filling the space.

The importance of the difference between the space required for the installation of the timber sets and steel supports can be inferred from *Figs. 49* and *50* which show these two types of support side by side in two large tunnels. *Fig. 51* illustrates the same difference in cross-sections.

When evaluated at the bid unit prices, the difference in quantities of excavation and concrete represents an important saving.

If the thickness of the concrete lining is determined by a specified factor of safety with respect to crushing, the quantity of both excavation and concrete can further be reduced, because a concrete shell reinforced by steel ribs can carry more per unit of thickness than a plain concrete shell.

Standards of comparison between timber and steel support

No direct comparison can be made between a timber and a steel beam. It cannot be said that a timber with so many square inches cross-sectional area is directly equal to a steel beam with such and such another sectional area. Nor can the section modulus, nor radius of gyration be used as conclusive criteria for evaluation.

In order to compare the relative merits of steel and wood, two supports must be designed for the same station of the tunnel, one of steel, the other of wood. Both must be able to carry the rock load with equal margin of safety when installed in the tunnel. When making the comparison all the items must be considered which are influenced by the construction material used for the support. The most important items are yardages of excavation and concrete, erection time and labor cost. The yardages can be compared by computation but the estimate of erection time and labor cost comparison is a matter of experience.

Position of and load on tunnel support

Steel ribs, as commonly used, are placed wholly within the design concrete, the outside flange being flush with the design concrete line. Timber sets, however, are usually placed outside the design line, with the inside face of the timber tangent to the design line.

According to *Chapter 5* the rock load per unit of width of the tunnel increases approximately in direct proportion to the width of the excavated space. The width of the excavated space for a tunnel with timber support is greater than that for the same tunnel with steel support. Hence the total load on the timber support is greater than that on the steel support due to both a higher unit load and greater span.

Thrust and bending moments in set members

The bending moments in the arch sections of both steel and timber supports depend on the spacing between the blocking points, which in turn depend on the rock

conditions. In this connection, distinction must be made between tunnel supports in good rock and in bad or heavy rock.

In *good rock*, timber sets do not require any blocking except at the panel points. Between these points the members of timber sets are straight, and, as a consequence, the bending moments are zero. Steel ribs are curved between blocking points. Therefore the loads which act on the rib at the blocking points produce bending moments. Theoretically this fact constitutes a disadvantage of the steel rib when compared to a timber set in good rock. In reality it merely reduces somewhat the savings resulting from the use of steel.

In contrast to good rock, *bad rock* requires either close blocking or else back packing, involving continuity of load transfer from the rock onto the ribs. If the blocks are closely spaced, the segments of timber sets are acted upon by rock loads not only at the panel points but also between these points. Since the timbers are straight between panel points, they carry their load as beams supported at both ends, and also must transmit thrust from adjacent segments. The maximum bending moment in these beams increases with the square of the distance between the panel points. A single blocking point between panel points is sufficient to require that about half the rock load be carried by beam action.

On the other hand, in steel ribs the bending moments decrease rapidly with increasing number of blocking points, and for continuous loading the bending moments are equal to zero. As the rock gets worse, the unit pressure on the tunnel support increases, but the spacing between the blocking points must be reduced. As a result, the bending moments in the steel ribs decrease in spite of the increase of the unit pressure, whereas the bending moments between the panel points in timber sets increase. Hence in bad rock the steel ribs perform at their maximum efficiency, because the bending stresses are very small compared to the compressive stresses, whereas the timber sets are subject to severe bending. The worse the rock, the greater is the economy resulting from use of steel.

Effect of moisture

The water conditions in the tunnel affect the allowable fibre stress in the timber. Most tunnels are at least damp, if not wet. Therefore the timbers are also moist if not wet. Moisture reduces the strength of timbers considerably, whereas the strength of steel is not affected by the presence of water or water vapor. Common practice is to reduce the allowable fibre stress where wood is "intermittently wet," and to make a further reduction for a "continuously wet" condition.

Allowable stresses

Whatever type of steel may be used as construction material for the steel ribs, the allowable stress for the steel has a definite value which can be ascertained in advance of construction. On the other hand, the allowable stresses for timber range between very wide limits. They are different not only for different kinds of timber, but also for different grades of the same kind. It is seldom practicable to specify in advance which kind and grade of timber should be used in a tunnel, whereas there can be no objection to specifying a steel with a definite strength. Hence the design of a timber support in advance of construction involves many uncertainties, whereas a steel support can be designed like a permanent structure, on the basis of figures which are well established.

Method of comparing timber and steel alternatives

According to the preceding discussions, the difference between the cost of tunneling with steel and timber support depends on many factors other than the price of the

construction material per unit of strength and unit of cross-section of the members of the sets. Therefore no comparison tables of general validity can possibly be prepared. The comparison must be made for every tunnel individually. The procedure is illustrated by the following example:

Comparison of steel and timber supports

Two typical tunnel supports are compared in *Fig. 51*, one side showing steel, the other, a timber support. The following data are to be used in preparing the designs and apply to both designs.

Fibre Stress

Steel, both compression and flexure, 24000 p.s.i.
Wood, flexure, 1400 p.s.i., compression, 1100 p.s.i.

Weight of Rock, assumed 170 lbs. per cu. ft.

Design Concrete Line is used in determining the percentages of excavation and concrete as this line is the limit of the useful construction.

Excess excavation and *excess concrete* indicate quantities beyond the design concrete line.

The quantities pertinent to the comparison are set forth in the following tabulation:

	Blocked *as used in Good Rock*		**Packed** *as used in Bad Rock*	
	Steel	*Timber*	*Steel*	*Timber*
Size of rib set	8″ x 8″ H @ 31 #	12″ x 12″	8″ x 8″ H @ 31 #	16″ x 16″
Spacing of rib set	4′ 0″	3′ 7½″	4′ 0″	2′ 6″
Spacing of Blocking Points	50″ Max.	Panel Points	0″	0″
Load Carried	19.8 ft. of Rock	19.8 ft. of Rock	25.5 ft. of Rock	25.5 ft. of Rock
Excavation per foot of tunnel to Design Line—				
in cu. yds.	17.7	17.7	17.7	17.7
percent	100%	100%	100%	100%
Excavation per foot of tunnel to overbreak line—				
in cu. yds.	20.1	22.7	20.1	23.3
Excess excavation	2.4	5.0	2.4	5.6
percent	13.5%	28.2%	13.5%	31.6%
Concrete per foot of tunnel to design line—				
in cu. yds.	4.2	4.2	4.2	4.2
percent	100%	100%	100%	100%
Concrete per foot of tunnel with volume of support deducted, in cu. yds.	to overbreak line 6.6	8.5	to lagging in arch, overbreak lines on sides 5.6	7.3
Excess concrete	2.4	4.3	1.4	3.1
percent	57.1%	102.4%	33.3%	73.8%
*Material in support structure per foot of tunnel	492 lbs.	255 fbm	492 lbs.	591 fbm

* This does not include lagging which would be about equal in both steel and wood structures.

Were it possible to chisel a tunnel out of intact rock, there would be no overbreak and no support would be required. In other words the tunnel could be built without spending any money on excess excavation and concrete. Hence any support supplied and any work done or material placed outside the design line represents an investment that adds no value to the completed structure. Therefore the relative economy of the two support materials can be evaluated by comparing the cost of support, excess excavation, and excess concrete combined. The following tabulation indicates the method:

	Steel	*Wood*
Steel, 492 lbs. @ ____ per lb., erected	_____	
Wood, 255 fbm @ ____ per 1000 fbm, erected		_____
Excess excavation, 2.4 cu. yds. @ ____ per cu. yd.	_____	
5.0 cu. yds. @ ____ per cu. yd.		_____
Excess concrete, 2.4 cu. yds. @ ____ per cu. yd.	_____	
4.3 cu. yds. @ ____ per cu. yd.		_____
Totals	_____	_____

The figures used in this tabulation are for the "blocked" condition of installation in so called "good" rock. Under the "packed" condition with severe loads, the saving effected by steel is considerably greater.

When the costs are computed at unit prices contained in balanced bids on tunnels of comparable size, the steel shows important savings over timber. Considering in addition, the faster progress which can be made when using steel support the engineer usually finds that steel supports are more advantageous than timber sets. He is therefore justified in showing steel supports in the plans and investigating timber later if submitted as an alternate by a bidder.

CONSTRUCTION PROCEDURE AND TYPE OF SUPPORT

The excavating cycle

In advancing a rock tunnel by blasting the usual sequence of operations is:

1. Drilling
2. Shooting—including clearing the tunnel, loading, and firing
3. Ventilating
4. Mucking
5. Erecting support

Shortening the elapsed time required for any one or all of these operations, speeds up the rate of driving, which in turn reduces the cost by reducing the gross overhead charges.

Next to the invention of the mechanically operated drill, and now, the modern boring machine, the advent of the drill carriage or jumbo was the biggest factor in speeding up tunnel driving. It very materially shortens the time necessary to set up the drills, permits a large number of holes to be drilled simultaneously, permits "full face" operations by providing working platforms at convenient levels, and in many cases telescopes erecting time of the support system into the drilling operation time. Jumbos are shown in *Figs. 52* and *53*.

Developments in ventilation and improvements in mucking and haulage equipment have also kept pace to contribute their share to more rapid tunneling.

Steel support, which can be erected in a fraction of the time required to erect timber sets, has made an important contribution to excavating speed.

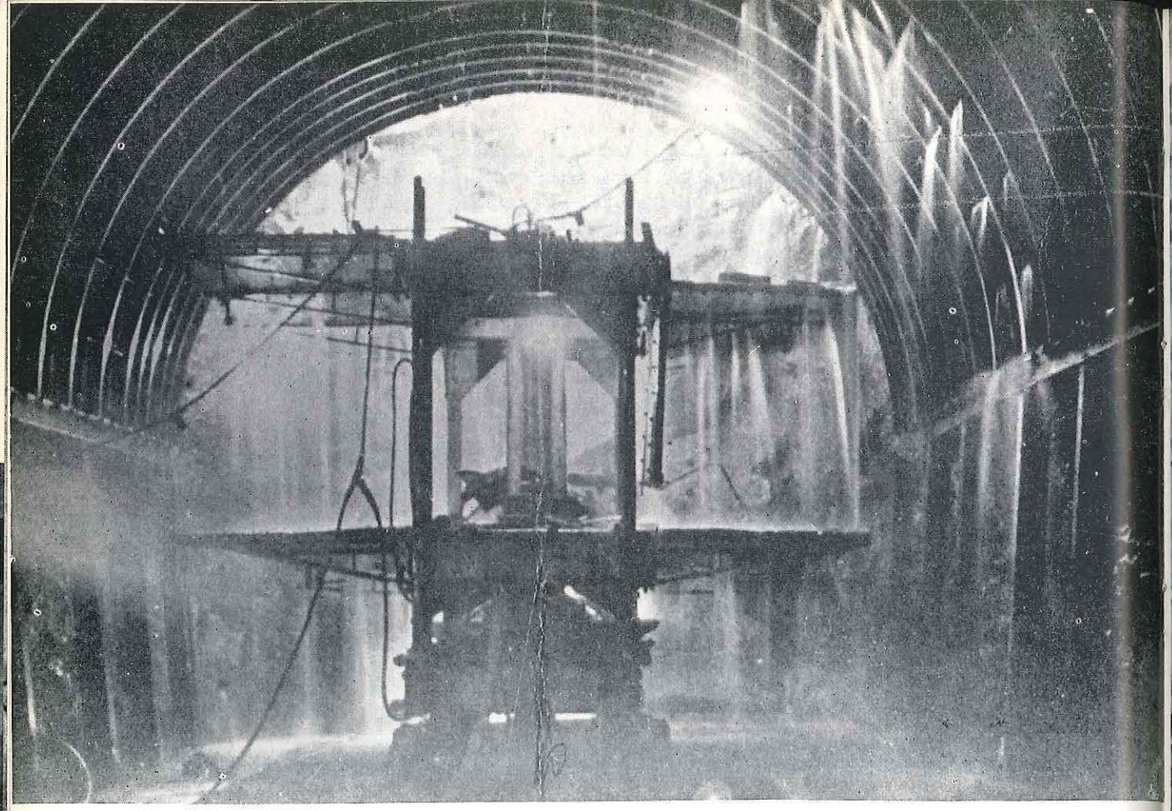


Fig. 52—Drill jumbo in a wet heading

The drill jumbo reduces time required for positioning the drills and provides access to all sections of the face so that a large number of holes can be drilled simultaneously. All supplies for the drilling operation are carried thereon. Ribs can also be transported to the face on the jumbo and erected easily from it.

Metropolitan Water District of Southern California

LOS ANGELES-COLORADO RIVER AQUEDUCT, SAN JACINTO TUNNEL
West Portal Heading, near Banning, California

Contractor: Force Account

Tunnel dimensions: driven bore, 19'-0" wide by 19'-2" high, horseshoe; finished bore, 16'-0" by 16'-0" horseshoe.

Support: rib, wall plate and post; ribs 6" H-beams, 25 lbs., at 16" and 32" centers; wall plate, single beam type, 6" H-beam, 25 lbs.

Lagging: 3/16" x 16" x 37-11/16" liner plates.

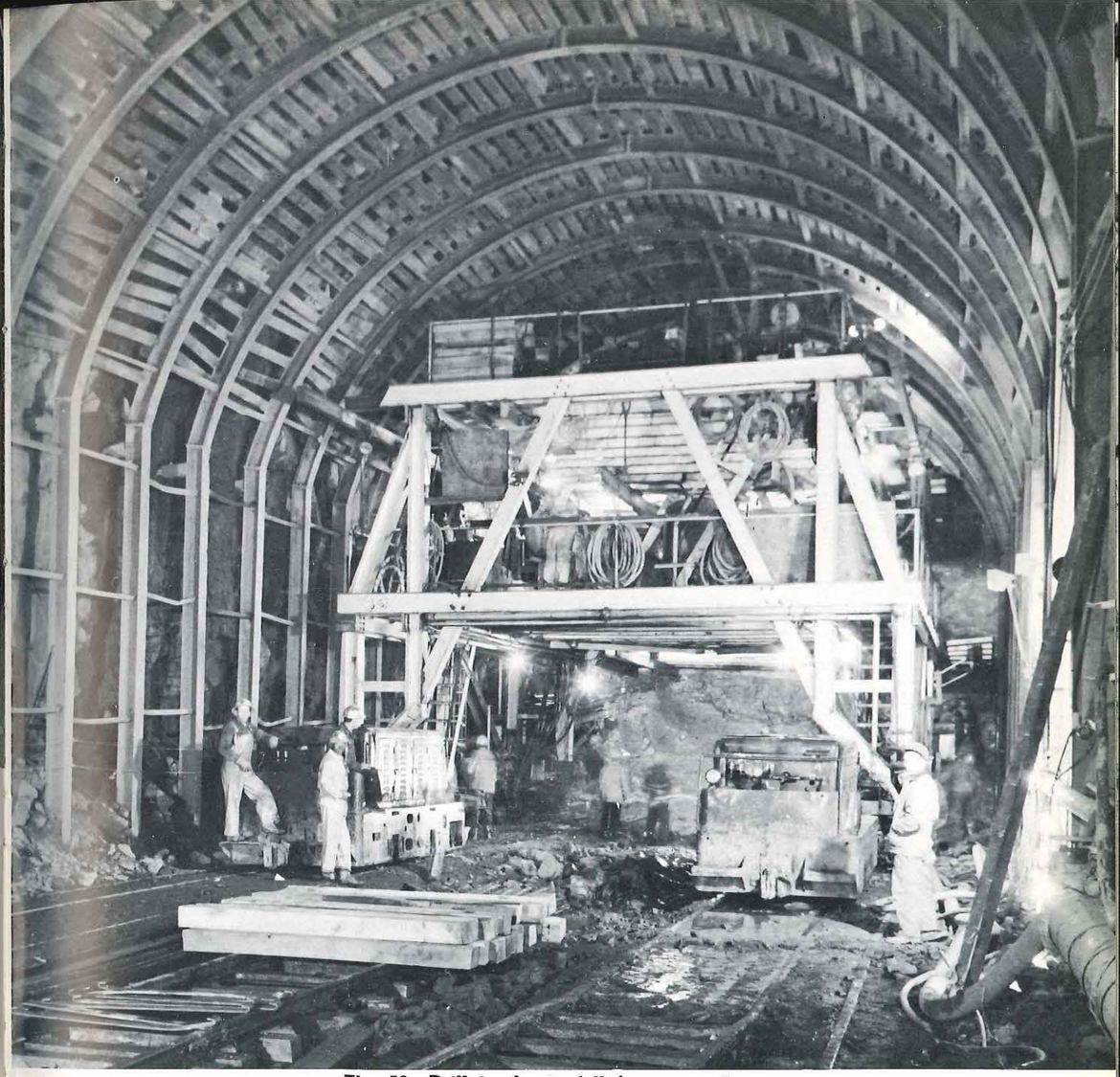


Fig. 53—Drill jumbo in full face operation

This rail mounted jumbo provides working platforms for erecting and blocking supports, installing lagging, and drilling holes for the next round. Arch supports can be carried to the heading on the upper platform. Because of its size this tunnel uses four piece ribs. Posts are set first from the floor, then arch ribs are set on top, held with two inner bolts and tipped into position. For smaller size tunnels two piece ribs are often used. The jumbo shown rides on rails, but many run on rubber tires or treads.

DOUBLE LANE HIGHWAY TUNNELS — KITTATINY MOUNTAIN, PENNSYLVANIA TURNPIKE

Contractor: C. J. Langenfelder & Son, Inc.

Tunnel Dimensions: *Driven bore, 36'-3" wide by 32'-11" high, straight sides. In finished tunnel flat ceiling separates vent area in arch from traffic area below.*

Support: *10" WF @ 33# spacing variable, 2'-0" to 4'-0" centers.*

Effect of bridge action period on sequence of operations

If the bridge action period is too short to permit mucking, it is necessary to change the sequence of operations and to support the roof prior to mucking. If the bridge action period is long enough to exclude the danger of important rock falls during the period of mucking, the normal cycle is adhered to and the support is usually erected from the drill carriage. Finally, if the bridge action period is considerably longer than a normal cycle, the support can be erected from another jumbo lined up behind the drill carriage, while drilling proceeds. This shortens the cycle quite considerably.

Factors determining method of attack

When choosing the support system the method of attack or method of excavation must be taken into consideration. The selection of the method always involves a compromise between an attempt to facilitate and accelerate the operation of mining and the necessity of supporting the rock before it starts to come down into the tunnel. Therefore the method of attack depends on the *rock behavior* and on the *size and shape of the tunnel cross-section.*

The methods of attack most commonly used are:

Full Face
Heading and Bench
Top Heading
Side Drift
Multiple Drift

Less frequently used are the *Top Drift,* the *Center Drift,* the *Bottom Drift,* and the *Bottom Heading* methods, but these are usually followed by enlargement to full size in one or in two operations. Hence, if one of these methods is used, the most suitable type of support depends not on the initial but on the final phase of the method of attack. This final phase can be the full face (one operation) or else the heading and bench or top heading methods (two operations).

Types of steel support systems

Rock tunnel support systems of steel are roughly of five types:

Continuous Rib	Rib and Wall Plate
Rib and Post	Rib, Wall Plate and Post

Full Circle Rib

These principal support systems are illustrated by *Figs. 54* to *58. Fig. 59* shows Invert Struts, in combination with continuous ribs.

Elements of steel tunnel supports

Every one of the steel tunnel supports listed above consists of two or more different elements, each of which serves a different function. Some of these elements, including the ribs, posts, wall plates, bracing and lagging, constitute part of the support system. Others, including the crown bars and truss panels, are temporary expedients for

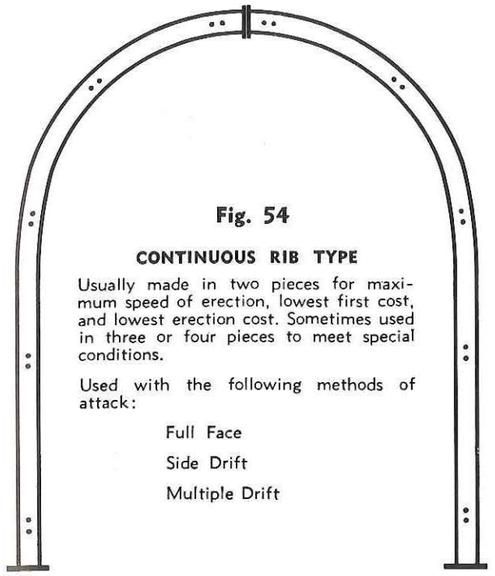


Fig. 54

CONTINUOUS RIB TYPE

Usually made in two pieces for maximum speed of erection, lowest first cost, and lowest erection cost. Sometimes used in three or four pieces to meet special conditions.

Used with the following methods of attack:

Full Face
Side Drift
Multiple Drift

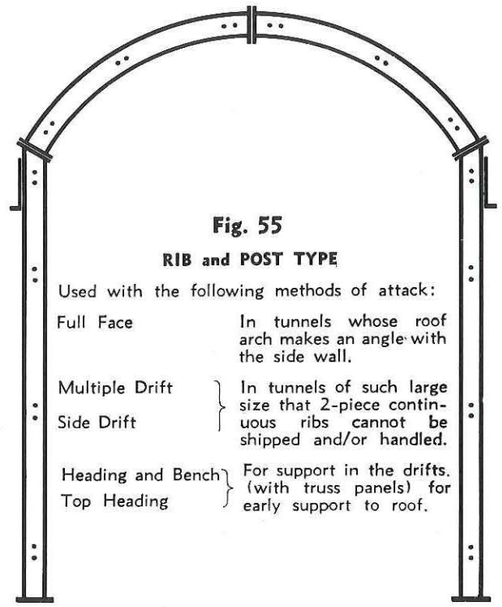


Fig. 55

RIB and POST TYPE

Used with the following methods of attack:

Full Face	In tunnels whose roof arch makes an angle with the side wall.
Multiple Drift, Side Drift	In tunnels of such large size that 2-piece continuous ribs cannot be shipped and/or handled.
Heading and Bench, Top Heading	For support in the drifts. (with truss panels) for early support to roof.

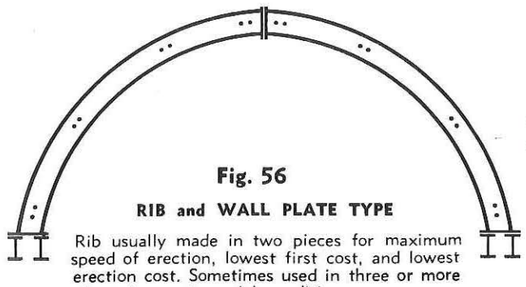


Fig. 56

RIB and WALL PLATE TYPE

Rib usually made in two pieces for maximum speed of erection, lowest first cost, and lowest erection cost. Sometimes used in three or more pieces to meet special conditions.

Used with the following methods of attack:

Heading and Bench
Top Heading
Full Face

This type is especially applicable to circular and high sided tunnel sections where only a light roof support is needed.

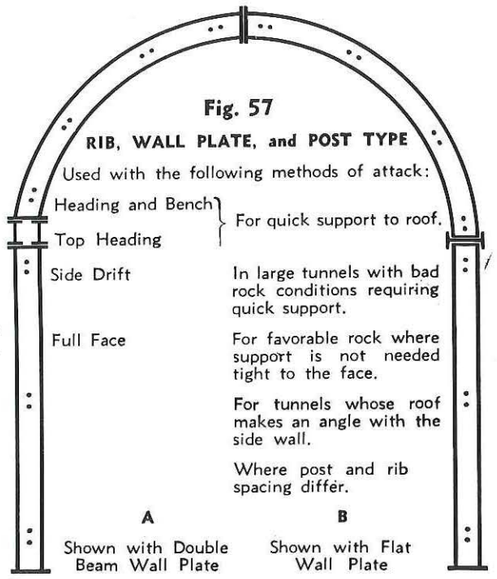


Fig. 57

RIB, WALL PLATE, and POST TYPE

Used with the following methods of attack:

Heading and Bench, Top Heading	For quick support to roof.
Side Drift	In large tunnels with bad rock conditions requiring quick support.
Full Face	For favorable rock where support is not needed tight to the face.
	For tunnels whose roof makes an angle with the side wall.
	Where post and rib spacing differ.

A	B
Shown with Double Beam Wall Plate	Shown with Flat Wall Plate

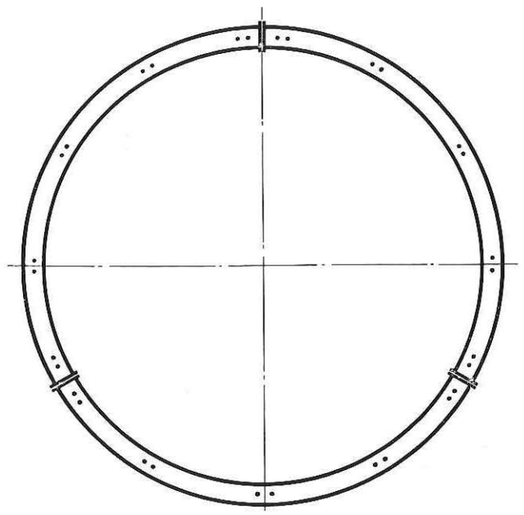


Fig. 58—Full circle rib type

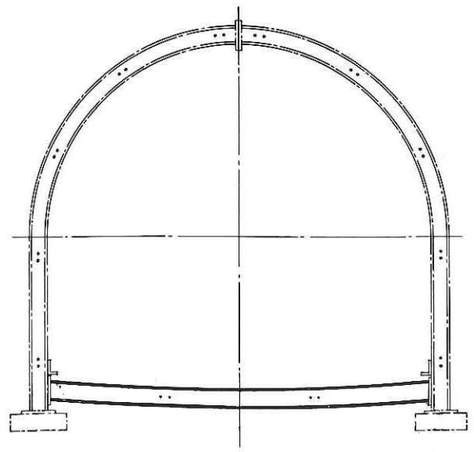


Fig. 59—Invert strut

Used where mild side pressures are encountered. Also used to prevent bottom from heaving.

Used with the following methods of attack:

Full Face	In tunnels in squeezing, swelling and crushed rock, or any rock that imposes considerable side pressure. Also where bottom conditions make it impossible to carry roof loads on foot blocks. In earth tunnel conditions sometimes encountered in rock tunnels.
Heading and Bench	Under earth tunnel conditions with joints at spring line.

supporting the rock while the tunnel support is being erected. To these elements must be added the blocks or back packing which transfer the rock load onto the tunnel support, and also the hardware such as bolts and nuts and various auxiliary equipment which is used as an aid during erection of the tunnel support.

The *Rib, Rib* and *Post* or *Rib, Post* and *Invert Strut* form a frame or "set" placed at right angles to the center line of the tunnel. The frames serve to receive the load and to transmit it to footings or else to carry it by ring action (as in full circle ribs).

The *Wall Plates* serve as sills for the ribs. They transmit the load from the ribs through blocks or posts onto the rock.

The *Lagging* bridges the space between the ribs and transmits rock load to the ribs.

The *Bracing* is required to prevent buckling or shifting of ribs or posts, unless this purpose is served by lags which are attached to the frame.

The *Crown Bars* are located in the crown of the tunnel, parallel to the center line. After shooting and ventilating, they can rapidly be slipped forward, to support the newly exposed roof by cantilever action beyond the ribs. They may also be used to support the roof temporarily while the bench is being taken out.

The *Truss Panels* serve a function similar to the latter function of the crown bars. They are located at the spring line and constitute a temporary support for the ribs while taking out the bench, to be replaced by posts in the final stage of erection.

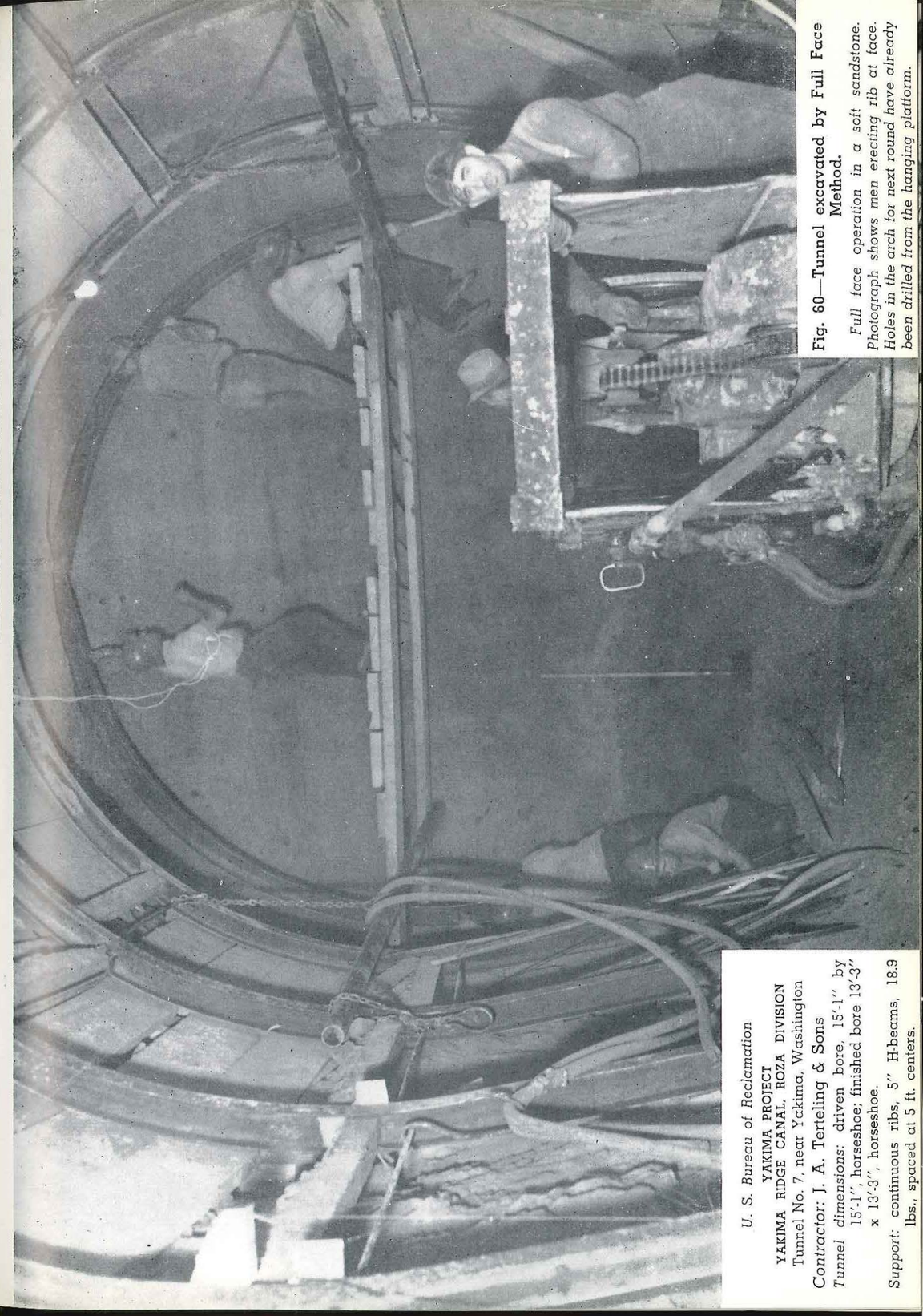
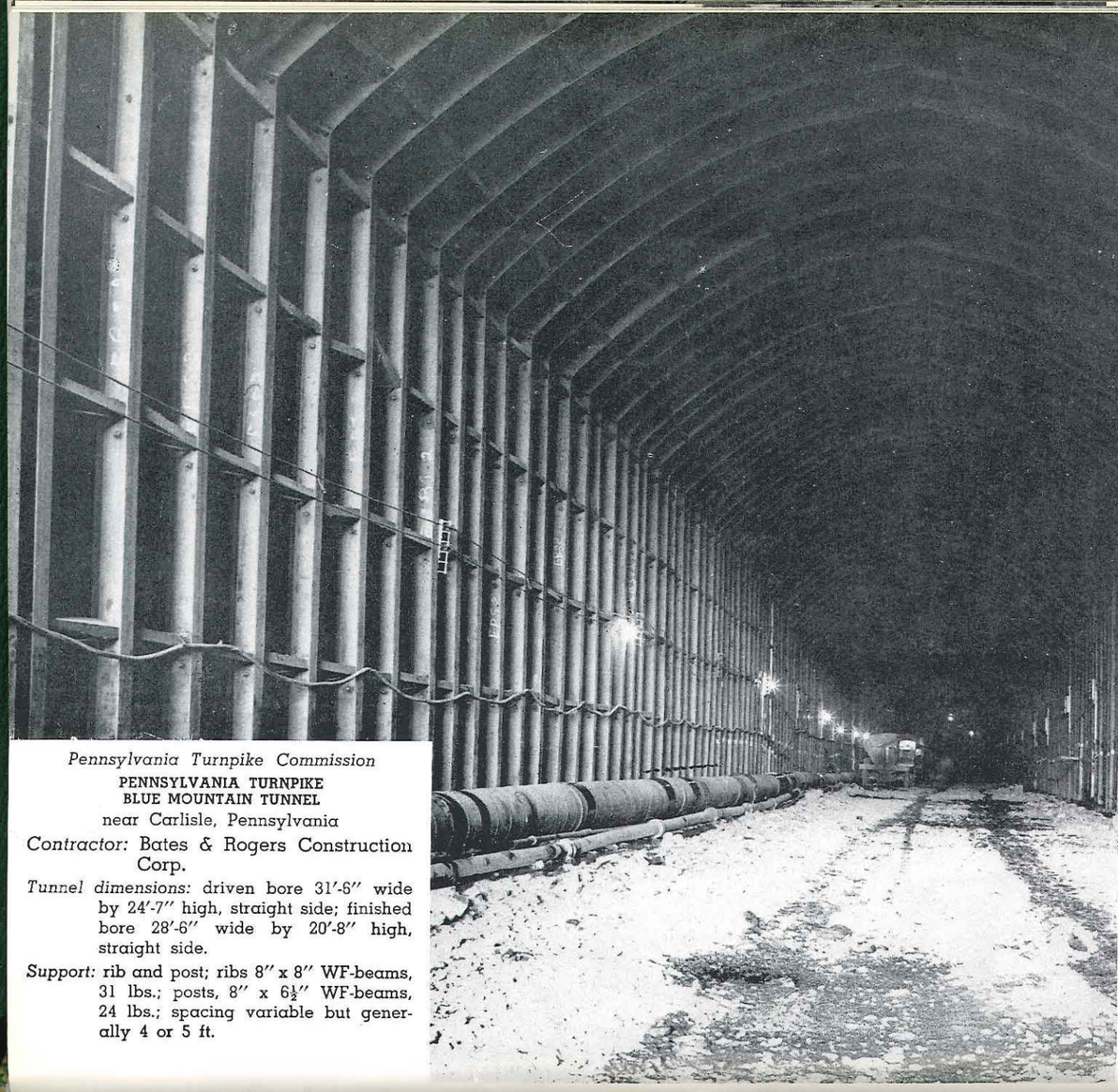


Fig. 60—Tunnel excavated by Full Face Method.

Full face operation in a soft sandstone. Photograph shows men erecting rib at face. Holes in the arch for next round have already been drilled from the hanging platform.

U. S. Bureau of Reclamation
YAKIMA PROJECT
YAKIMA RIDGE CANAL, ROZA DIVISION
Tunnel No. 7, near Yakima, Washington
Contractor: J. A. Terteling & Sons
Tunnel dimensions: driven bore, 15'-1" by 15'-1", horseshoe; finished bore 13'-3" x 13'-3", horseshoe.
Support: continuous ribs, 5" H-beams, 18.9 lbs., spaced at 5 ft. centers.



Pennsylvania Turnpike Commission

PENNSYLVANIA TURNPIKE
BLUE MOUNTAIN TUNNEL

near Carlisle, Pennsylvania

Contractor: Bates & Rogers Construction Corp.

Tunnel dimensions: driven bore 31'-6" wide by 24'-7" high, straight side; finished bore 28'-6" wide by 20'-8" high, straight side.

Support: rib and post; ribs 8" x 8" WF-beams, 31 lbs.; posts, 8" x 6½" WF-beams, 24 lbs.; spacing variable but generally 4 or 5 ft.

Fig. 61—Rib and Post type of steel support.

The Rib and Post type of support was required as the roof arch meets the side walls at an angle and the dimensions were too great for the two-piece continuous ribs.

There are seven two-lane highway tunnels on the "Turnpike" six of which are enlargements of abandoned railroad tunnels. None of the six had been holed through, however, when work stopped on the railroad project. "Rib and Post" type of steel support was used in enlarging and completing the seven tunnels on this project.

Factors determining choice of support system

When choosing the support system the following factors must be considered:

Method of attack,
Rock behavior, and
Size and shape of the tunnel cross-section.

To these factors must be added the limitations which are imposed upon the size of the individual members of the tunnel support, by the capacity of the means available for the transportation of the members from the factory to the job, and by the space which is available in the tunnel for manipulating the members into their final position.

The following text contains a description of the principal methods of attack, of the types of steel support suitable for each, and the considerations influencing the selection of type of support.

Full face method

General Procedure. In a full face operation, the tunnel is blasted out full size at each round. Small size tunnels always were driven full face. Large tunnels were driven by "Heading and Bench," or some other method, due to difficulty of mounting drills on posts. The drill carriage, giving quick and easy access to all parts of the face no matter how big, made full face the standard method, to be departed from only where rock behavior and the load-time factors dictate otherwise. By far the greatest footage of tunnel has been constructed by this method. A full face operation is shown in *Fig. 60.*

This method is applicable to rock whose bridge action period is long enough to permit ventilating and mucking.

Types of support. Wherever possible the *Continuous Rib* type of support is used. In its 2-piece form, *Fig. 60,* it is cheapest and can be erected more rapidly than other types.

In a tunnel whose roof joins the side walls at an angle instead of a smooth curve, the *Rib and Post* type would ordinarily be used. See *Fig. 61.* There is nothing to prevent the roof rib and post being welded together at the spring line to make a two-piece set, but commonly the tunnel is of such size that the resulting rib could not be shipped or handled. The erection advantages of the two-piece rib can be secured by bolting the roof rib and the post together before erecting, if desired.

The *Rib and Post* type is commonly used in large tunnels, such as double-track railroad or two-lane highway tunnels, to keep the size of the rib segments within handling and shipping limitations.

In tunnels with a large cross-section with high straight sides through good rock or in large circular tunnels, the *Rib and Wall Plate* type is frequently used, as shown in *Fig. 62,* as it is less expensive to set the wall plates in hitches or on pins at spring line than to bring the steel down to subgrade or working floor elevation. This type of support is also satisfactory for mining through spalling rock provided spalling occurs only in the roof. It is also usable in some stratified and even broken rock. However, in many cases it is extremely difficult to establish adequate support for the wall plate at any point above the floor line due to irregularity of the overbreak. Where heavy loads are encountered, as in some broken rock and decomposed rock, this type of support cannot be used as the loads must be carried on down to firm footings.

oot of Shaft 15, near Mt. Kisco, New York

ontractor: Frazier-Davis Construction Co. *Subcontractor* to S. A. Healy Co.

nnel dimensions: driven diameter, 19'-2"; finished diameter, 15'-0".

largement: width, 32'-0" out to out of wall plates.

pport: rib and wall plate, ribs 6" H-beams, 25 lbs, spaced at 4 ft.; wall plate, double beam type, 8" I-beams, 18.4 lbs.

gging: 6" channels, clamped to ribs.

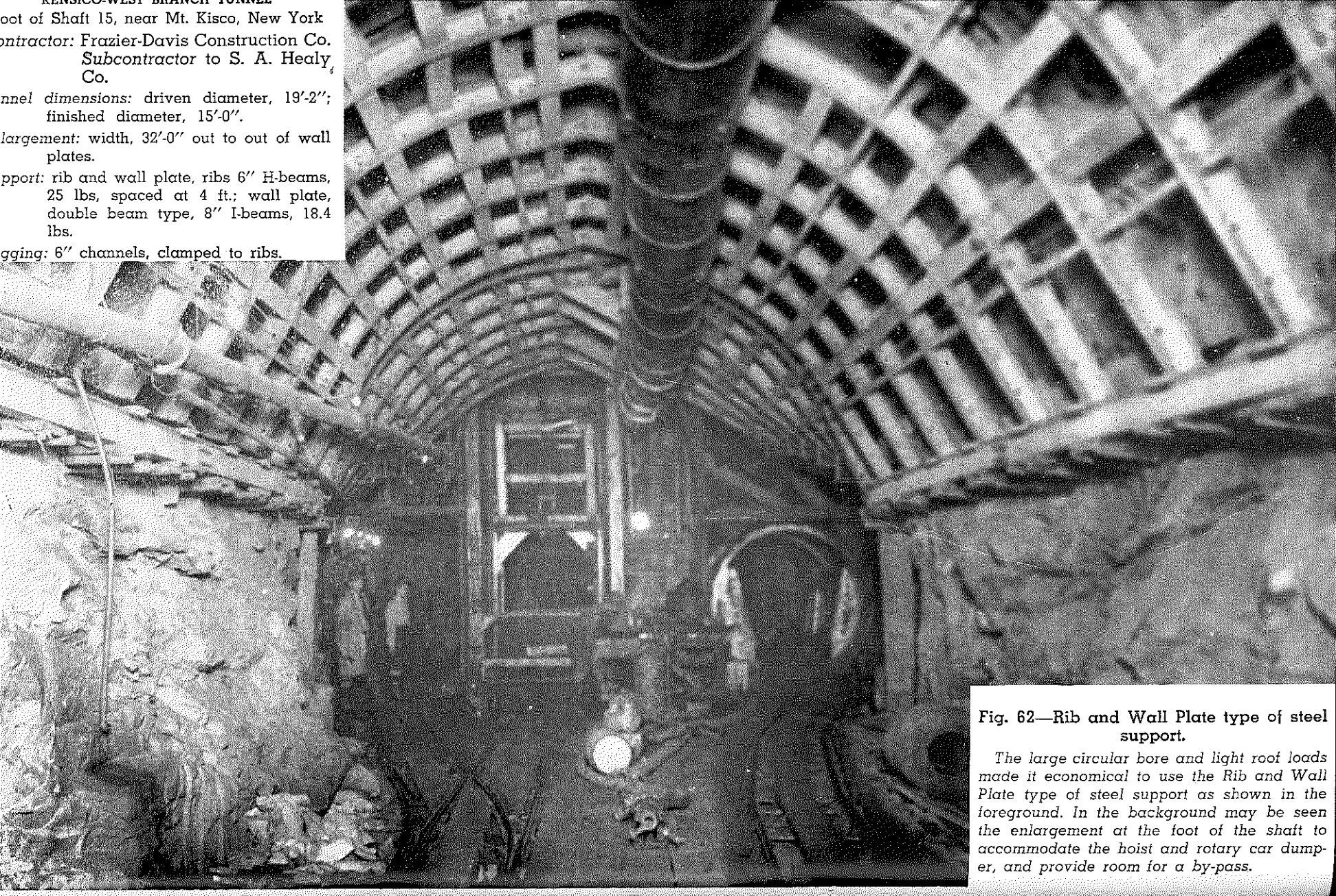


Fig. 62—Rib and Wall Plate type of steel support.

The large circular bore and light roof loads made it economical to use the Rib and Wall Plate type of steel support as shown in the foreground. In the background may be seen the enlargement at the foot of the shaft to accommodate the hoist and rotary car dumper, and provide room for a by-pass.

EAST 79th STREET WATER TUNNEL

in Chicago, Illinois, going west from Lake Michigan

ntractor: S. A. Healy Company. .

nnel Dimensions: Driven bore 15'-1" wide by 16'-0" high straight sides.

pports: Arch Ribs and posts, 6" H @ 20#; wall plates, 8" I @ 23# double beam type. Lagging 6" structural channels.

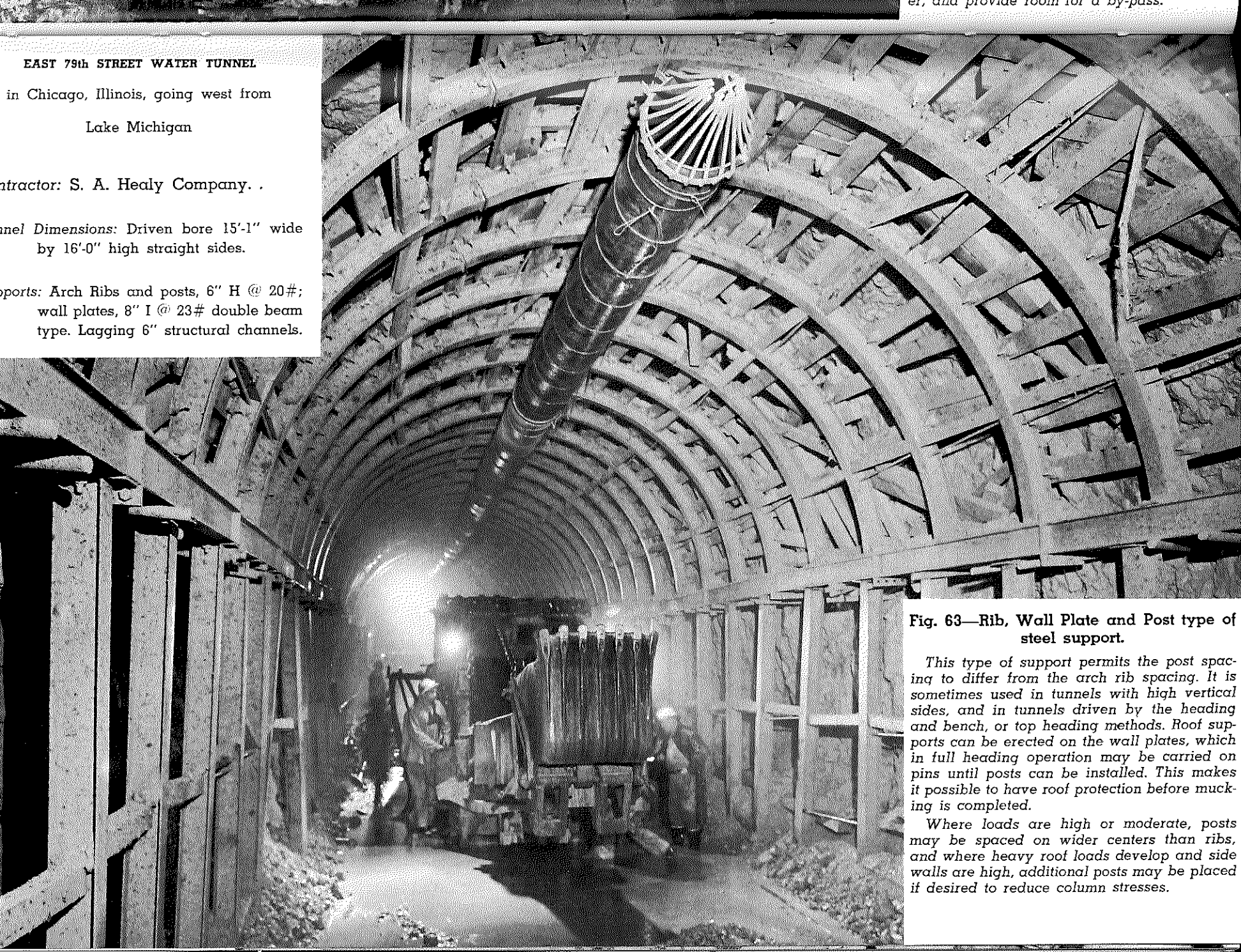


Fig. 63—Rib, Wall Plate and Post type of steel support.

This type of support permits the post spacing to differ from the arch rib spacing. It is sometimes used in tunnels with high vertical sides, and in tunnels driven by the heading and bench, or top heading methods. Roof supports can be erected on the wall plates, which in full heading operation may be carried on pins until posts can be installed. This makes it possible to have roof protection before mucking is completed.

Where loads are high or moderate, posts may be spaced on wider centers than ribs, and where heavy roof loads develop and side walls are high, additional posts may be placed if desired to reduce column stresses.

Fig. 64—Invert struts

This photograph shows invert struts used to resist side and bottom pressure in squeezing rock. The steel rib sets with invert struts were originally installed in this fault zone without knee braces. Space between ribs was gunited flush with the ribs. Excessive side pressures caused failure of a few invert struts. These were replaced with heavier sections (the first three struts in the foreground) and the whole series both new and old were reinforced with knee braces.

The geologists' report on this stretch of tunnel is reproduced in Fig. 13, page 44.

U. S. Bureau of Reclamation

COLORADO-BIG THOMPSON PROJECT, ADAMS TUNNEL THROUGH CONTINENTAL DIVIDE
West Portal, near Granby, Colorado

Contractor: Platt Rogers, Inc.

Tunnel dimensions: driven bore, 11'-9" wide by 12'-1" high, horseshoe; finished diameter, 9'-9".

Support: continuous ribs and struts, 6" H-beams, 22.5 lbs. spaced at 1 ft. to 5 ft. centers.

• • •

In large tunnels with high vertical sides in rock which has such a long bridge action period that steel support can be erected behind the drill carriage, the *Rib, Wall Plate and Post type* can be used to advantage. In this case, footings for the posts, often of concrete, can be prepared in advance. The posts are erected and capped by flat wall plates, which are merely a construction expedient. They form grooves into which the arch rib feet can be rapidly placed, without bolts. They also provide a convenient surface for horizontal blocking to take thrust.

The Rib, Wall Plate and Post type preferably using the "Double Beam Wall Plate," may also be used in large vertical-side or horseshoe shape tunnels where roof loads are light and the post spacing can be greater than the rib spacing as shown in *Fig. 63.* The converse is also true, where roof loads are so heavy that more posts than ribs are required to carry the load.

By means of the Rib, Wall Plate and Post type of support, sometimes supplemented by crown bars, the full face attack can be carried out even if the rock requires support before mucking. Immediately after ventilating, the roof ribs are erected on wall plates.

• • •

Fig. 65—Rib and invert strut failure

Failure was due to overloading in the fault zone. Leg has failed under combination of vertical loads and horizontal pressures. The deformation of the flange at the foot of the rib is indicative of the intense side pressure. The reverse bend in the middle of the strut indicates heaving of the bottom or subsidence of the ends and consequent settling of the rib set.

The ribs fail in the unworked straight leg. The Full Circle Rib type of support has been devised to meet pressure conditions that cause such failures as this. The full circle rib is not subject to bending and the working strength of the steel has been increased by cold forming. (See article on Improvement of Steel by Cold Working in Chapter 9.)

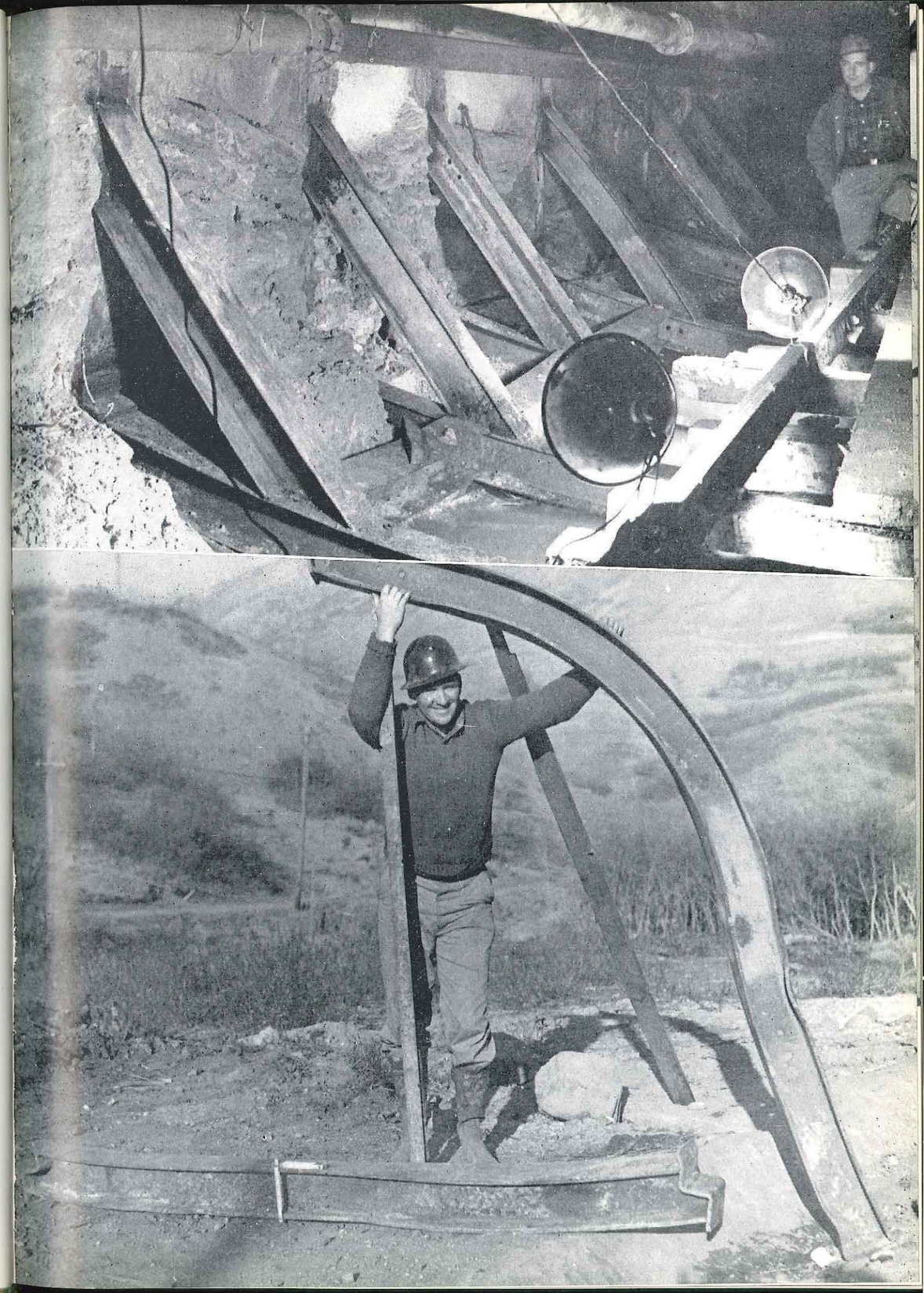
U. S. Bureau of Reclamation

PROVO RIVER PROJECT, ALPINE-DRAPER TUNNEL
near Provo, Utah

Contractor: Thompson-Markham Co.

Tunnel dimensions: driven bore, 8'-8" wide by 8'-3¼" high; finished diameter, 6'-10".

Support: continuous ribs and invert struts, 6" H-beams, 27.5 lbs.; spacing variable. Other sizes and types of support also used.



supported on pins projecting from holes drilled in the rock. While this support cannot take heavy loads, it can support the key stone of the half-dome until mucking is complete. The posts are installed under the wall plates while drilling is in progress for the next round.

In crushed and swelling rock the *Full Circle Rib* type should be used. Where it is possible to convert the outside of the tunnel to a full circle, the rib steel performs at its highest efficiency. It resists pressure from any or all directions, top, sides, or bottom, equally well. A more detailed description of the full circle support is contained in *Chapter 8,* dealing with the methods for mining through squeezing or swelling rock. However, some tunnels, such as railroad or highway tunnels are not so easily converted to the full circle. Hence, because of their high sides other provisions must then be made for resisting side pressure and preventing a heave of the bottom. The most common one is the installation of the *Invert Strut.* These are placed across the bottom of each set to resist the side pressure by providing a horizontal reaction for the post. Being curved, the strut converts the side pressure to a downward pressure on the bottom thus preventing a heave. Invert struts are shown in *Figs. 64* and *65.*

In such rock the posts of a high side tunnel perform also as vertical beams. They carry the heavy vertical loads as columns and in addition may have to resist sizeable horizontal pressures as beams. Hence they must have a large cross section and be closely spaced.

Heading and bench method

In this method, illustrated by *Fig. 66,* a top heading is carried ahead of the bench about 1½ times the length of one round, usually about 6 to 16 ft. The heading has the full width of the tunnel and is carried down to the spring line. This method was standard for large tunnels prior to the advent of the drill carriage, as it provided a working surface on which to mount the drills for drilling the arch section of the face.

Since then, however, it is used only where the bridge action period of the rock is so short that support is required immediately after blasting and ventilating. In such conditions it has two advantages: (1) Having the bench as a convenient work platform, temporary posts can be quickly placed under rock which shows signs of falling, or crown bars may be slid forward to hold the roof, before starting to erect steel. See *Fig. 80,* page 149; (2) Mucking can proceed simultaneously with the erection of roof steel in the heading.

Both bench and heading are shot out at each round, the bench charges being fired first. The heading shots throw most of the heading muck out onto the bench muck pile, which is under the protection of the steel erected after the preceding round was fired. As soon as the air is clear, the roof is scaled down to drop any fragments which threaten to fall soon. Muck is cleared at the sides to make room for the wall plates and the steel support is then erected. The remaining muck is pushed out onto the bench muck pile, whereupon drilling for the next round can proceed.

The type of steel support is limited to one which includes a wall plate, either "Rib and Wall Plate," or "Rib, Wall Plate and Post." The wall plate functions as a sill to carry the roof ribs, and in turn is supported by blocks and short posts, usually of wood, in the hitch. If steel posts to floor level are being used, the wall plate also functions to bridge the bench shot until the posts are installed after mucking.

The "Rib, Wall Plate and Post" type may be supplemented by *Truss Panels* or Crown Bars, which are accessories developed to handle heavy loads that come quickly,

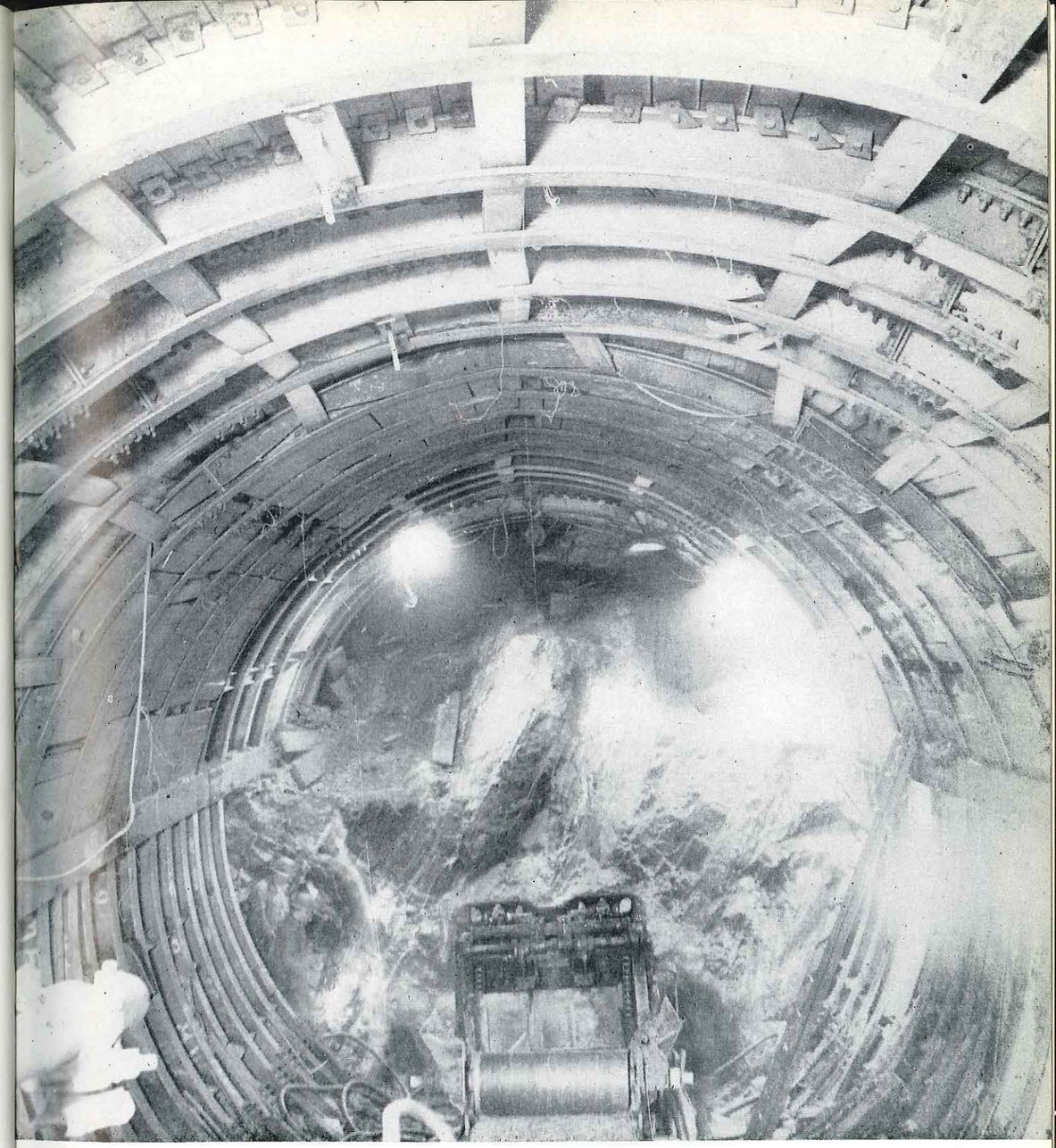


Fig. 66—Heading and Bench method

The heading and bench method is used where rock conditions will not permit full face operations. In this illustration the tunnel is being driven through a fault zone with full circle support, flat wall plates and wall plate drifts. The four planks at the spring line at the left partially conceal the wall plate drift.

Note: The appearance of this heading two weeks earlier is shown in Fig. 42, page 92.

Board of Water Supply, New York DELAWARE AQUEDUCT, KENSICO-WEST BRANCH TUNNEL

Contractor: S. A. Healy Company. North Heading from Shaft 16 near Armonk, New York

Tunnel dimensions: driven diameter, 24′-4″; finished diameter, 15′-0″.

Support: full circle ribs, 8″ x 8″ WF-beams, 67 lbs., six piece; spacing in roof, 4′-0″, below spring line, 2′-0″ average.

Lagging: 6″ channels, clamped to ribs.

Fig. 67—Top Heading method

Due to bad roof conditions caused by cemented gravel in some portions and thin strata of sandstone separated by immature shale in other locations, the top heading was holed through before starting to take out the bench. The Rib, Wall Plate and Post type of support was used.

The photograph shows workmen installing posts under the wall plates. Posts at the left have been placed. Only the post next to the bench at the right is in position. The wall plate is bridging the space where three more posts will be installed.

Note: The Knights tunnel on this same project, and located only a few hundred yards away, was driven "Full Face." The arch was located in a thick layer of sandstone which made roof conditions much better. See fig. 16 a, page 50.

U. S. Engineer Photo

TYGART RIVER DAM PROJECT, R. R. LOCATION, LANE TUNNEL
near Grafton, W. Va.

Contractor: Guthrie-Marsch-Walker Company.

Tunnel dimensions. driven bore, 21'-10" wide by 30'-8" high, straight side; finished bore 18'-0" wide by 27'-0" high, straight side.

Support: rib, wall plate and post type; ribs and posts, 8" x 5¼" WF-beams, 17 lbs. Rib spacing 2 and 4 ft. centers; post spacing variable; wall plate, single beam type, 8" H-beam, 32.6 lbs.

Lagging: 3/16" x 24" x 48" liner plates.

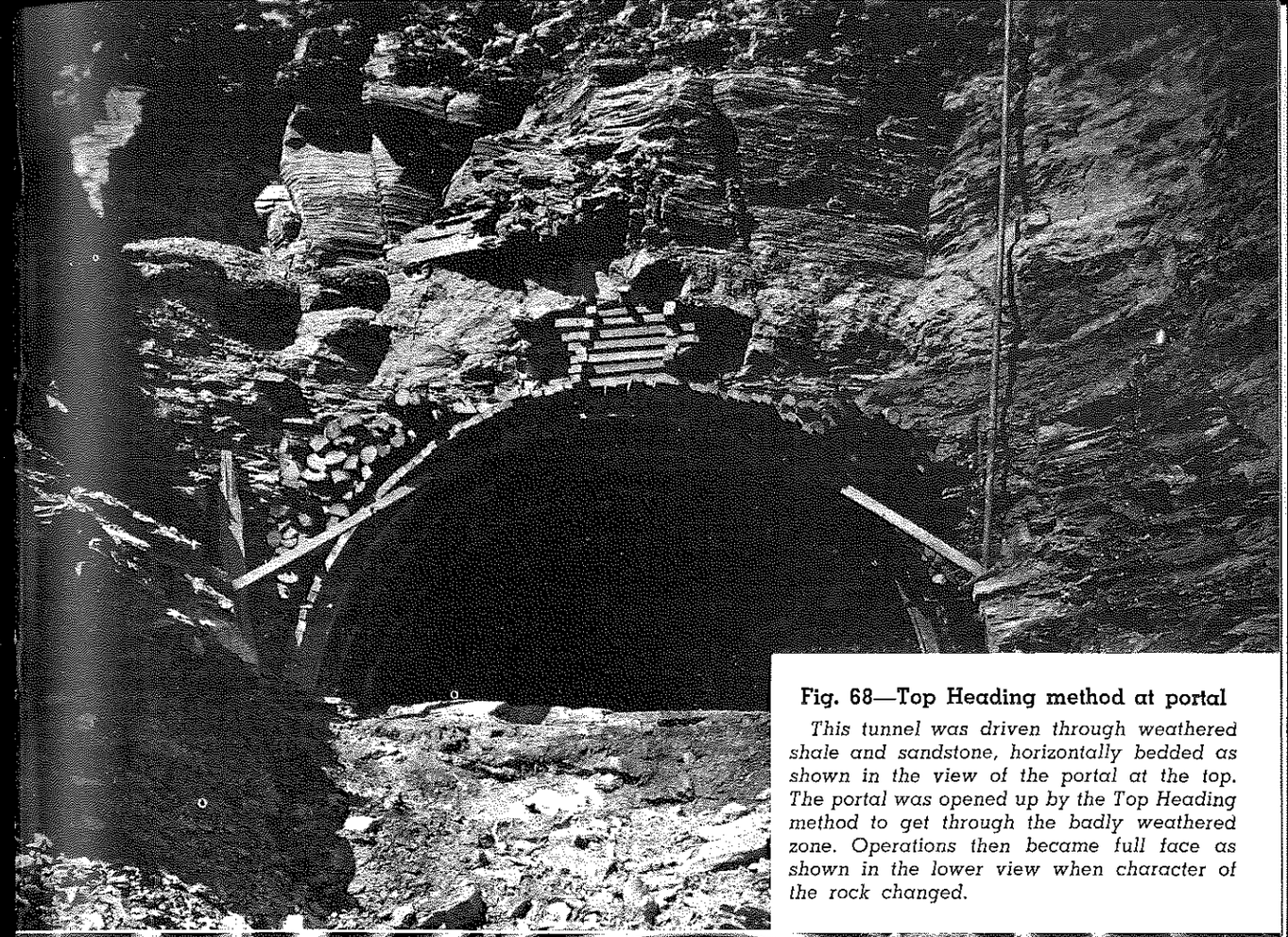
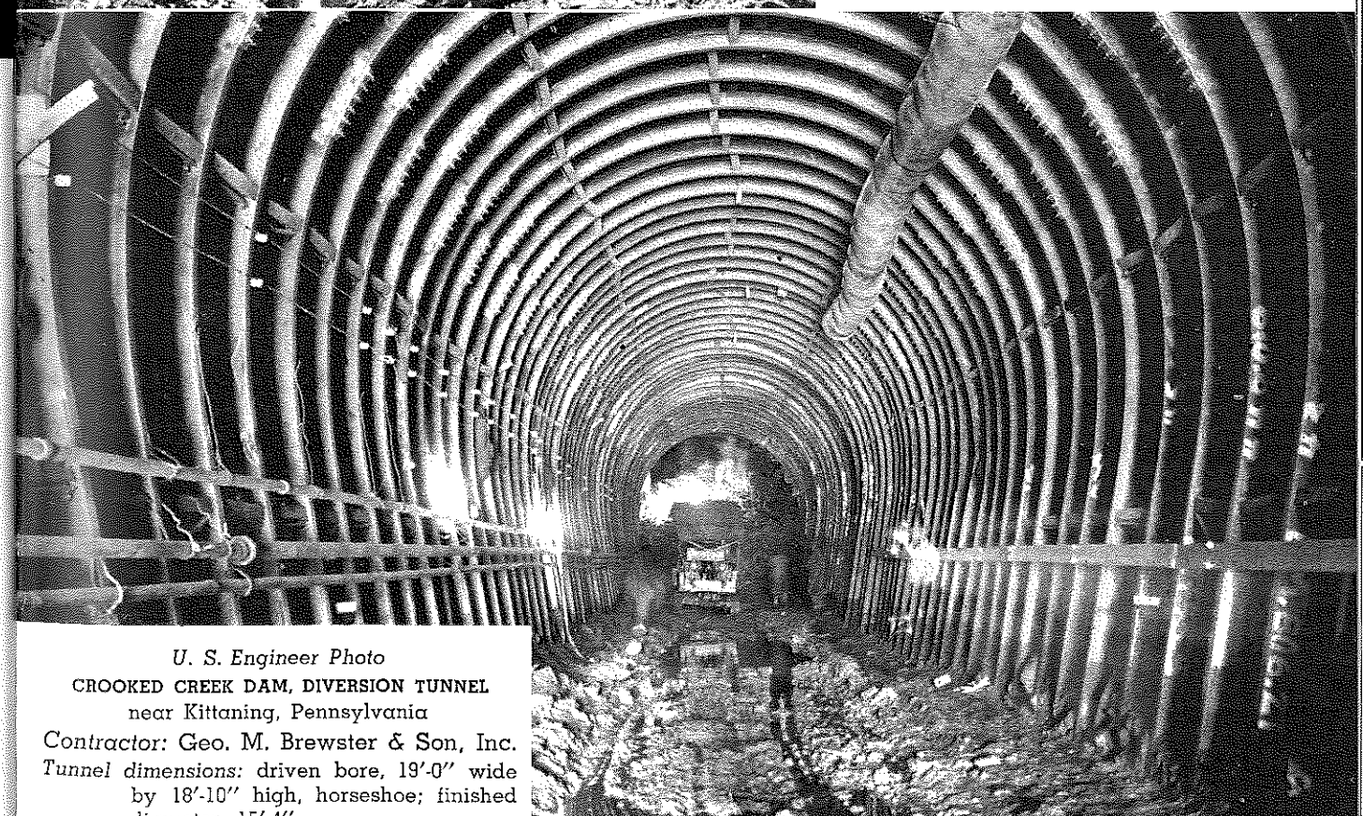


Fig. 68—Top Heading method at portal

This tunnel was driven through weathered shale and sandstone, horizontally bedded as shown in the view of the portal at the top. The portal was opened up by the Top Heading method to get through the badly weathered zone. Operations then became full face as shown in the lower view when character of the rock changed.



U. S. Engineer Photo

CROOKED CREEK DAM, DIVERSION TUNNEL
near Kittaning, Pennsylvania

Contractor: Geo. M. Brewster & Son, Inc.

Tunnel dimensions: driven bore, 19'-0" wide by 18'-10" high, horseshoe; finished diameter, 15'-4".

by supporting the intervening ribs while the bench is shot out. Their use is described later (see *Chapter 7).*

Top heading

Instead of taking out the bench along with the heading the top heading may be driven clear through as one operation, followed later by removal of the bench as another operation as illustrated in *Figs. 67* and *68*. The "Rib and Wall Plate" type of support is used in the heading, and posts may or may not be used when the bench is taken out.

This method is applicable where bad roof conditions are known to exist in most of the tunnel. One of the main advantages of this method is that trouble is met and overcome in the smaller portion of the excavation, and when ready, the major portion of the excavation can be done faster, and without delays due to bad roof. There is also a powder economy over the full face method, as holes can be drilled vertically to shoot the bench. A disadvantage is that the roof loads have had a long time to build up and when the bench is taken out, considerably more load must be transferred to the posts than in the "Heading and Bench" method. In some cases crown bars may be needed in addition to the wall plates (see *page 151).*

Wall plate drift

Both "Heading and Bench" and "Top Heading" methods are sometimes supplemented by drifts at each side on the spring line, advanced beyond the heading face to receive wall plates. These drifts are driven where the rock is so bad that only a short shot can be pulled in the heading. The purpose is to permit use of wall plates of sufficient length to equal or exceed the length of the subsequent bench shots. See *Fig. 66,* page 131.

Side drift

The "Side Drift" method of attack is sometimes employed in a large size tunnel (such as a two-lane highway or a double-track railroad tunnel) through bad rock which requires support before mucking out. The "Rib, Wall Plate and Post" type of support is used, the wall plate being in the form of the "Flat Wall Plate." A drift is driven ahead at each side at subgrade wherein the posts and wall plates are erected. See *Fig. 69.*

If the rock is not too bad, the operation may become full face from here on, as the wall plates will be exposed above the muck pile and the roof ribs can be quickly erected thereon before mucking out. Where extreme conditions are encountered, however, breakups to the crown are made, leaving a central core. If necessary, temporary posts may be quickly placed between the core and the roof at dangerous spots, or crown bars may be slid forward to quickly catch up the roof.

The roof ribs can then be placed on the wall plates and securely blocked to take the roof load, whereupon the temporary posts may be removed. If the working space is too constricted to permit mechanical handling of the segments of the conventional 2-piece set, ribs composed of several segments are commonly used.

The core may then be removed, or left in until after concreting as a support for the forms.

The side drifts themselves usually need support, which is removed just prior to shooting out the core of the main tunnel, and is reused ahead. The support system used

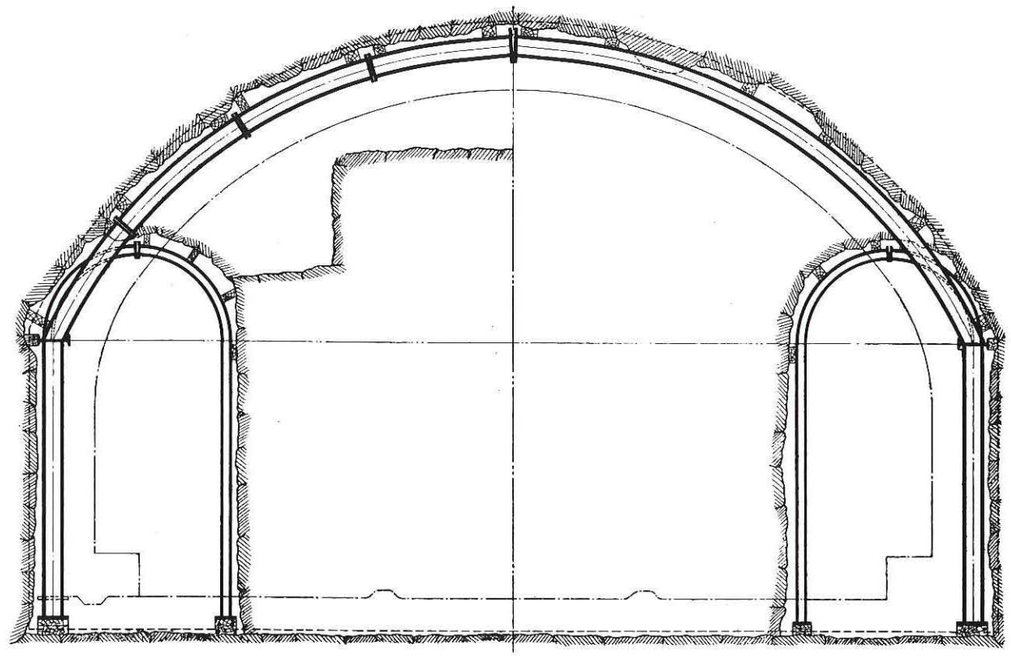


Fig. 69—Side drift method

The side drifts are driven ahead of the main excavation for some convenient distance. The support of these drifts includes the main tunnel support posts and wall plates. Just prior to shooting the main bore, the drift support is removed, leaving the main posts and wall plate in position. These project above the muck pile, thus permitting erection of the main arch ribs before mucking out.

a)—Where rock conditions are bad, "break ups" to the crown are made, leaving the core in place. The core provides a convenient surface from which to quickly support the roof by jacks or posts till the ribs can be placed. The ribs are usually multi-piece for man handling in the restricted space.

b) — Where the rock conditions permit, the main tunnel excavation is done full face. The ribs are usually in the 2-piece form for speedy erection and early support to the main roof. Crown bars may be used to catch up the roof till the ribs are placed.

for the drifts is a hybrid. The outer side consists of the posts and wall plate which later become a part of the support for the main tunnel, whereas the inner side is a "Continuous Rib."

A variant of the side drift method is to drive the drifts clear through and to make them large enough to construct that portion of the permanent concrete lining which is below the spring line. The main tunnel ribs are installed as an arch on these concrete abutments. The main support and the drift support are of the "Continuous Rib" type in this case.

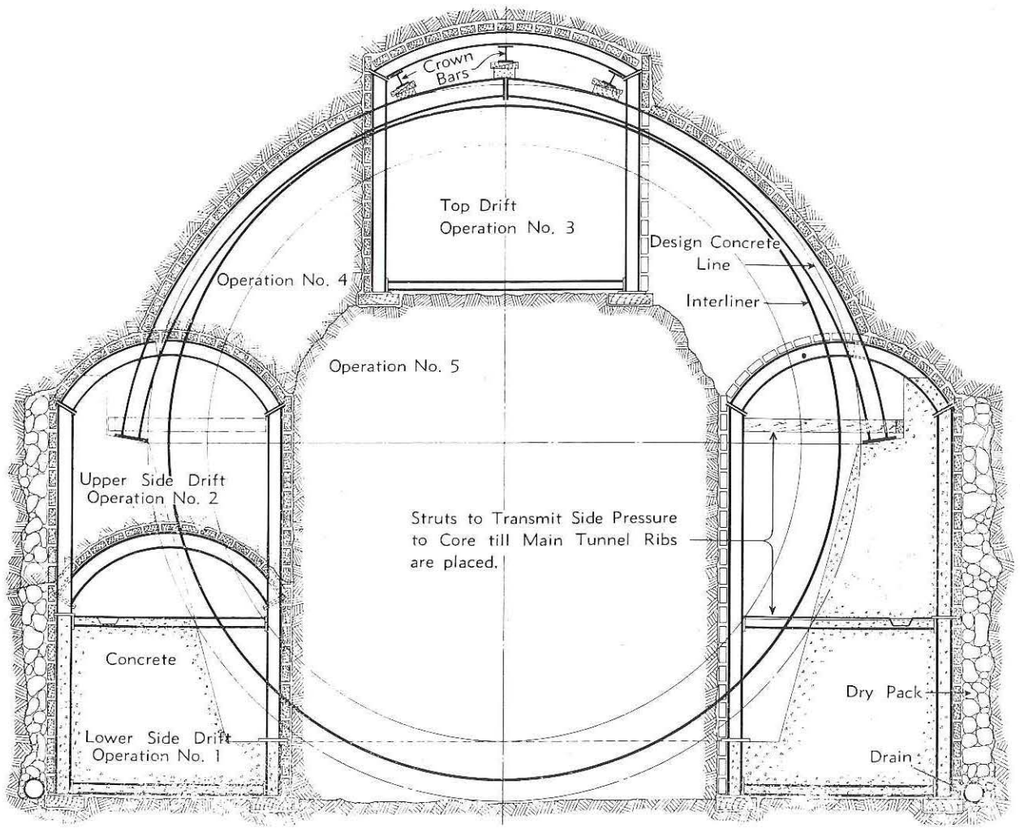


Fig. 70—Multiple drift method

Multiple drift method

This method is usually a combination of side drifts and top drift, borrowed from earth tunnel practice. It is frequently employed to get through crushed rock in fault zones which may behave like earth, even if the rock is compacted enough to require light blasting. See *Fig. 70.*

A side drift is driven through the fault at subgrade on each side. A concrete side wall is placed in each drift, with adequate provisions for drainage. Should the height of the side walls be too great to build in a single drift, another side drift may be driven immediately above, and the concrete side walls carried on up to spring line as shown in *Figs. 70* and *41,* page 90.

A top center drift is then driven through, with the roof support far enough above the future position of the main tunnel ribs to provide space for crown bars over the ribs. A short section of the drift roof is blocked on the crown bars and the drift side posts are removed.

The top drift is widened out by means of short shots to connect with the roofs of the side drifts. The main arch ribs are erected on the concreted side walls, lagged

and packed. The crown bars supporting the roof members of the top center drift are securely blocked to the ribs, whereupon the next advance can be made.

The support for the main tunnel is the "Continuous Rib" type, usually in 2-piece form although it may be made of more than two pieces. The type of support for the drifts may be the two-piece "Continuous Rib" or the "Rib and Post" type.

Other methods of attack

As mentioned previously in this Chapter, there are other methods of attack. These involve drifts or headings at different positions in the tunnel bore which may be desirable for a number of reasons other than any connected with steel support. For instance, it may be desirable to drive a bottom drift ahead of the main excavation to explore and drain the rock ahead, or to increase the number of working faces.

Or a center drift may be driven to explore the rock and permit breakups to the crown in bad stretches which can be supported in advance, thus avoiding slowdowns of the main excavation. Powder economy may be a consideration, as blasting from radial holes parallel to the plane of the face is more effective than from holes normal to the face. The Fort Peck Dam Tunnels were constructed with a center drift as shown in *Fig. 47,* page 106.

A center drift is highly desirable in swelling rock, to permit a relaxation of the stresses in the rock and to collect reliable information on the pressures exerted by the rock.

Inasmuch as all these other methods are eventually superseded by the Full Face or the Top Heading method as soon as enlarging starts, no discussion is needed. The main tunnel support is not installed until the process of excavation has arrived at either one of these two stages. The drift would ordinarily be supported by the 2-piece "Continuous Rib," or the "Rib and Post" type. The "Full Circle Rib" type is most desirable for a drift through squeezing or swelling ground.

Design of tunnel support

After the type of support has been selected for the principal rock conditions which are likely to be encountered in a proposed tunnel, the contract drawings for the supports are prepared. Earlier in this chapter the elements of which the different tunnel supports are composed were listed. These elements can be divided into three groups. The first group contains the ribs, posts and invert struts which are oriented at right angles to the center line of the tunnel. They transmit the rock load onto rock-supported blocks or footings. The second group includes the wall plates, crown bars, truss panels and the bracing. Since all of them are located lengthwise, parallel to the center line of the tunnel, they constitute the longitudinal members of the tunnel support. The third group includes the lagging, which bridges the spaces between the ribs.

In order to design a steel tunnel support in accordance with the requirements of the job, the engineer must first of all be familiar with the types and with the functions of each element. These are described in the following *Chapters 7* and *8. Chapter 7* deals with the elements of tunnel support to be used under normal conditions; *Chapter 8* with tunnel supports required when mining in soft or swelling ground.

The last step in designing a tunnel support is a stress computation, to make sure that the stresses in the members of the tunnel support are within allowable limits. The methods of stress computation are covered by Section III.

CHAPTER 7

CONSTITUENTS OF TUNNEL SUPPORTS

TRANSVERSE MEMBERS

Factors determining layout of rib systems

The ribs serve to transmit the rock load onto footings or sills. Therefore the required strength of the entire rib system is determined by the rock load. However, for a given load and given cross-section it is necessary to decide two points: *first*, the spacing between the ribs, and *second*, whether the ribs should be ordered in two pieces or whether it is necessary to use multi-piece ribs. The following paragraphs deal with the factors which should be considered when making these decisions.

Rib spacing

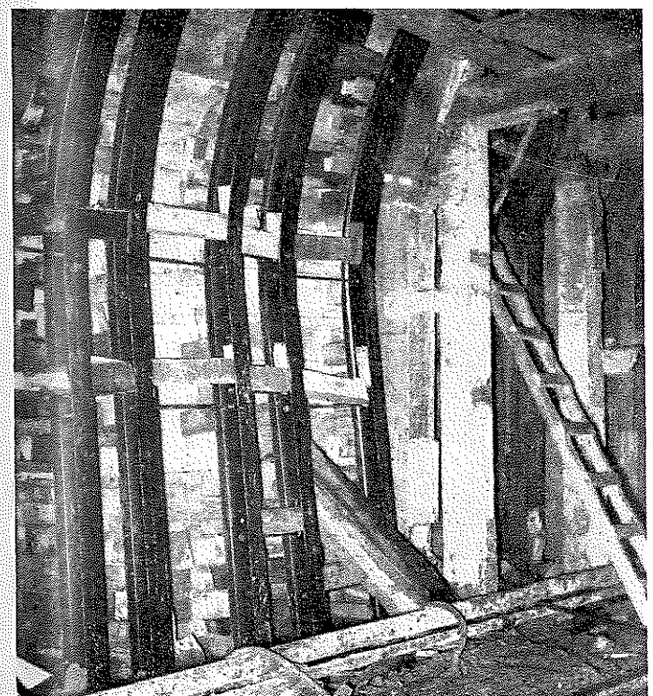
Heavy ribs cost less per pound, and consequently less per unit of strength, than light ribs. The shop cost of fabricating a heavy rib is only slightly more than for a

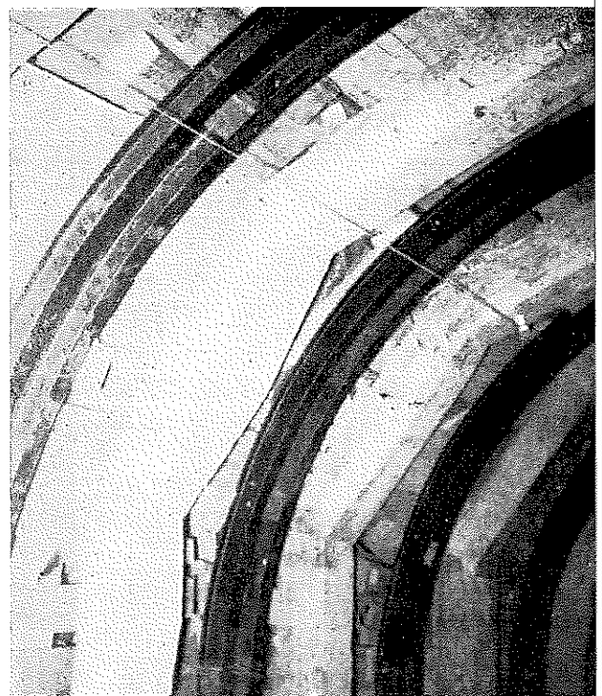
Fig. 71—Jump sets installed to relieve overloaded supports

Twin-rib jump sets installed between timber sets in heavy ground. At the left, two timber sets in foreground have been removed. The rectangular reinforcement for the segmental timber sets in background was removed after steel ribs were placed.

Metropolitan Water District of Southern California
LOS ANGELES-COLORADO RIVER AQUEDUCT WEST IRON MOUNTAIN TUNNEL
near Rice, California

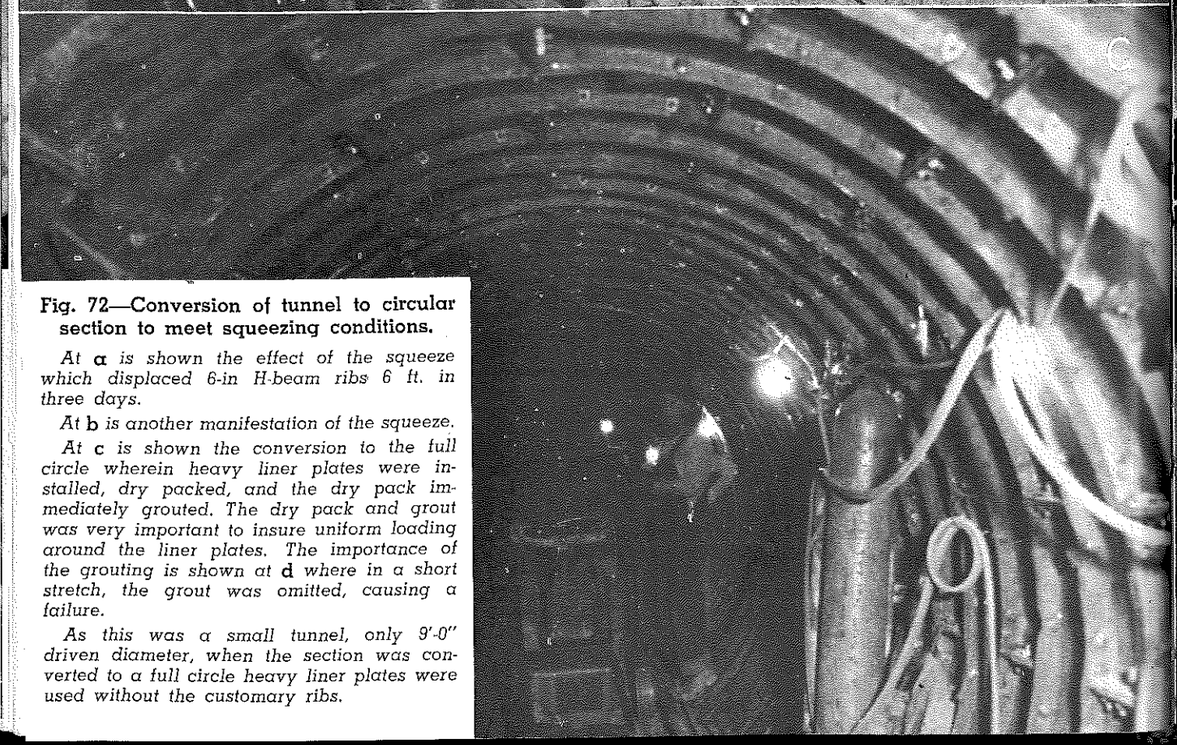
Contractor: Utah Construction Co.
Tunnel dimensions: driven bore, 19′-0″ wide by 18′-8″ high, horseshoe; finished bore 16′-0″ by 16′-0″, horseshoe.
Steel support: continuous ribs, for jump sets, 6″ I-beams, 12.5 lbs.



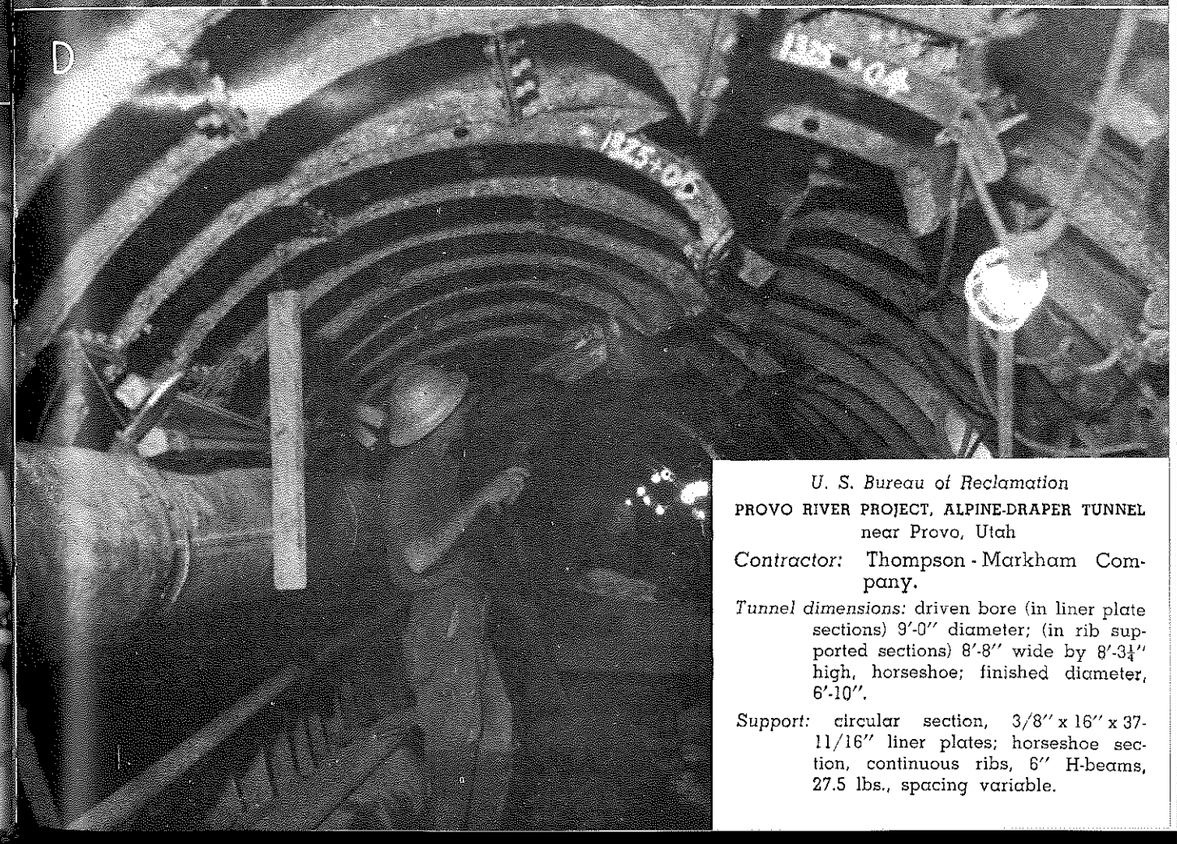


A

B



C



D

Fig. 72—Conversion of tunnel to circular section to meet squeezing conditions.

At **a** *is shown the effect of the squeeze which displaced 6-in H-beam ribs 6 ft. in three days.*

At **b** *is another manifestation of the squeeze.*

At **c** *is shown the conversion to the full circle wherein heavy liner plates were installed, dry packed, and the dry pack immediately grouted. The dry pack and grout was very important to insure uniform loading around the liner plates. The importance of the grouting is shown at* **d** *where in a short stretch, the grout was omitted, causing a failure.*

As this was a small tunnel, only 9'-0" driven diameter, when the section was converted to a full circle heavy liner plates were used without the customary ribs.

U. S. Bureau of Reclamation

PROVO RIVER PROJECT, ALPINE-DRAPER TUNNEL
near Provo, Utah

Contractor: Thompson - Markham Company.

Tunnel dimensions: driven bore (in liner plate sections) 9'-0" diameter; (in rib supported sections) 8'-8" wide by 8'-3¼" high, horseshoe; finished diameter, 6'-10".

Support: circular section, 3/8" x 16" x 37-11/16" liner plates; horseshoe section, continuous ribs, 6" H-beams, 27.5 lbs., spacing variable.

light one. The erection cost per rib is about the same, heavy or light. At a wider spacing a greater length of tunnel is supported, hence the rib cost is less.

On the other hand, with increasing spacing the free span of the lagging increases. Since the lagging transmits the load from the rock onto the ribs, the required strength and the cost of the lagging increases with rib spacing. Hence the spacing should be chosen such that the sum of the cost of ribs and of lagging is a minimum. Experience indicates that this condition is satisfied if the spacing is about 4 ft. for moderate rock loads, 2 to 3 ft. for heavy rock, and 5 ft. for very light loading. With these figures in mind the spacing must be adapted to the special requirements of the job. Thus, for instance, in rock which requires support tight to the face the spacing should be an even fraction of the length of the shot pulled. If safety niches, junction boxes, ventilating flues or other appurtenances are to be incorporated into the concrete lining, the spacing should be chosen such that the ribs are located between them. In other cases, the spacing is determined by the size of prefabricated lags, or by the dimensions of the concrete forms, because the construction joints should be located between the ribs.

Required strength of ribs

The required strength of the ribs is determined by the estimated rock load and the spacing. The estimate of the rock load is based on geological data. In *Chapter 5* it has been shown that the geological data often leave a wide margin for interpretation, because the character of fault zones and the presence or absence of zones of hydrothermal alterations can never be predicted with certainty. Furthermore, the rock conditions always change to some extent from place to place. Even if a tunnel crosses a ridge containing only one type of rock, the load on the tunnel support changes as the tunnel passes from the zone of surface weathering into the less altered core of the ridge, and the importance of the change can only be guessed at in advance of construction. Hence when studying his tunnel problem the engineer should always think in terms of probabilities and not of certainties. Both the most favorable and most unfavorable possibilities need to be considered.

Demonstrated by examples in *Chapter 5*, the range of loading for any given rock in different conditions is extremely wide. Hence it is quite obvious that one assumption of loading would be uneconomical. To assume too light a loading leads to use of ribs which fail in the load increase period. "Jump sets" installed to relieve failing ribs, as shown in *Fig. 71*, cost far more in extra ribs and tunnel labor than the extra cost of ribs of adequate strength put in at the original installation. Furthermore, when it becomes apparent, during the load increase period, that the ribs are too light, the future spacing will inevitably be reduced, with the attendant higher cost per foot of tunnel. Conversely, to assume a loading that approaches the worst condition leads to use of unnecessarily heavy ribs in long stretches of tunnel.

The extremes of roof loading for the various rock conditions (where no side pressure is anticipated) can be tabulated. On the basis of this tabulation an assumption can be made of an intermediate loading that may reasonably be expected to prevail in the major portion of the tunnel. The next step is to choose the normal rib spacing in accordance with the rock conditions (4 to 5 ft. for light, 4 ft. for medium, and 2 to 3 ft. for heavy loads) and to select a rib which can carry the estimated load per rib. This rib constitutes the "standard" rib. The standard rib would safely carry all loads lighter than the intermediate loading, but for reasons of economy a "light" rib should also be selected. To sustain the heavier loads a "heavy" rib is selected. These ribs can be shown on the tunnel cross sections.

When the geological data or local experiences indicate the presence of fault zones, or zones of advanced chemical alteration of the rock, one should be prepared to cope with extra heavy loading and heavy side pressures. Crushed rock, squeezing rock, and swelling rock create such conditions. The most effective way to deal with them is to convert the tunnel to a full circle, even though very considerable quantities of extra excavation and concrete may be required to maintain inside clearances. *Fig. 72, a* to *d* illustrate such conditions which were met by conversion to a circular section.

In *Chapter 5* it was mentioned that reliable data concerning the pressure exerted by swelling rock are very scarce. The pressures discussed in that chapter were estimated on the basis of what was observed when timber sets failed and the records of the failures contained hardly any information on the amount of inward movement of the rock which produced the failures. However, by using a full circle rib the engineer can make sure he is getting the maximum results from each pound of steel in the ribs.

The bending stresses in properly installed full circle ribs are almost zero and since the ribs are cold worked in all portions, their strength is much higher than ordinary structural steel. (See *Chapter 9*).

The wise engineer takes one additional step. He provides in his specifications for an initial supply of each weight of rib and further provides that a reasonable reserve stock of each will be maintained.

Spacing changes versus weight changes

As the loading changes from one portion of the tunnel to another, it is more desirable to change the weight of the steel rib than to change the spacing, as indicated

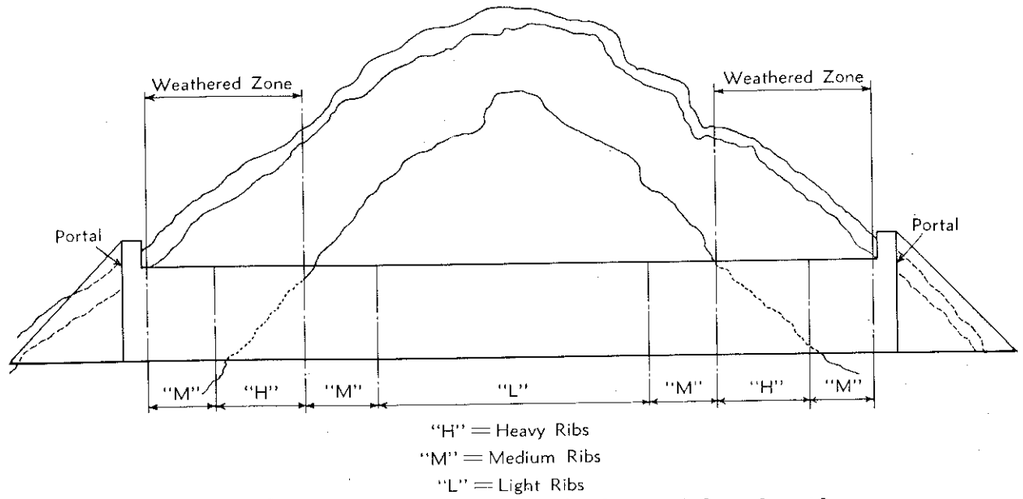


Fig. 73—Proper use of various weight ribs in tunnel through a ridge

in *Fig. 73*. If supplies of the various weights are on hand, this change can be made without trouble. Bracing and lagging lengths remain constant. But if only one profile or weight of steel is available, the spacing must be changed, with the attendant trouble and expense of changes in bracing and lagging. With ribs of several weights on the job most conditions can be met, but a spacing change is still possible to cope with emergencies.

Rib cross-section and depth

Ribs generally are fabricated of structural beams. H-beams or Wide Flange beams are preferable to I-beams as the wider flanges provide more generous surfaces for blocking and lagging, and the section has greater resistance against twisting. Channel sections are not suitable as their unsymmetrical section invites twisting, and their flanges are too narrow. In small tunnels, however, channels bent about their minor axis are quite acceptable under ordinary loads as in *Fig. 127*, page 206.

When choosing the profiles with different weights it is usually advisable to select beams of equal depth. This avoids complications associated with changes in pay lines, clearances, etc.

Length of rib sections

In general, the fewer the joints in a rib set, the lower are the costs of material and of the labor involved in erecting the rib. In this respect the two-piece set deserves the preference over the multi-piece sets. However, the members of two-piece sets for large tunnels are very long and their chord depth is great. In order to find out whether 2-piece ribs with very large dimensions are practicable, both the shipping and the handling limitations to size and weight must be considered.

Rail or truck transportation imposes certain definite limitations on the physical dimensions of ribs. When these limits are reached, it is necessary to divide rib sets into three or four pieces, or a "Continuous Rib" type of support may have to be changed to a "Rib and Post" type.

The shipping limitations must be taken for granted and can hardly be changed. On the other hand, the handling limitations depend to a large extent on the layout of the job. By properly planning the job it may be possible to avoid the necessity of using the more expensive multi-piece set and to install two-piece sets instead. Hence the handling limitations deserve careful attention in the layout stage of the project. Thus, for instance, proper provisions should be made for the transportation of the rib sections within the tunnel, and on the drill carriage or "jumbo" for erection. If driving a tunnel from a shaft, provision should be made in the shaft and skip, and especially at the "eye" (or connection from shaft to tunnel) for getting the longest ribs into the tunnel. If consideration is given to this at the time of laying out the job this provision can be satisfied at little or no extra expense.

Posts

The posts serve to transmit the rock load from the arch ribs to footings on the bottom of the tunnel. The spacing between the posts is commonly equal to that of the ribs. However, by inserting between the ribs and the posts a stiff longitudinal beam (wall plate), the spacing of the posts can be made independent of that of the ribs.

Since the posts act as columns, they are as a rule made of H-beams. The depth of these beams is commonly the same as that of the ribs though in many cases they can be of a lighter section as long as no side pressure is present. If side pressure acts on the posts they must be proportioned to act as vertical beams as well as columns.

Invert struts

Where side pressures are present and the tunnel section has not been converted to a full circle, it is necessary to prevent the inward movement of the rib or post feet. Side pressure is often accompanied by an upward movement of the bottom and more frequently by a "soft bottom" which will not sustain the load imposed by the footings.

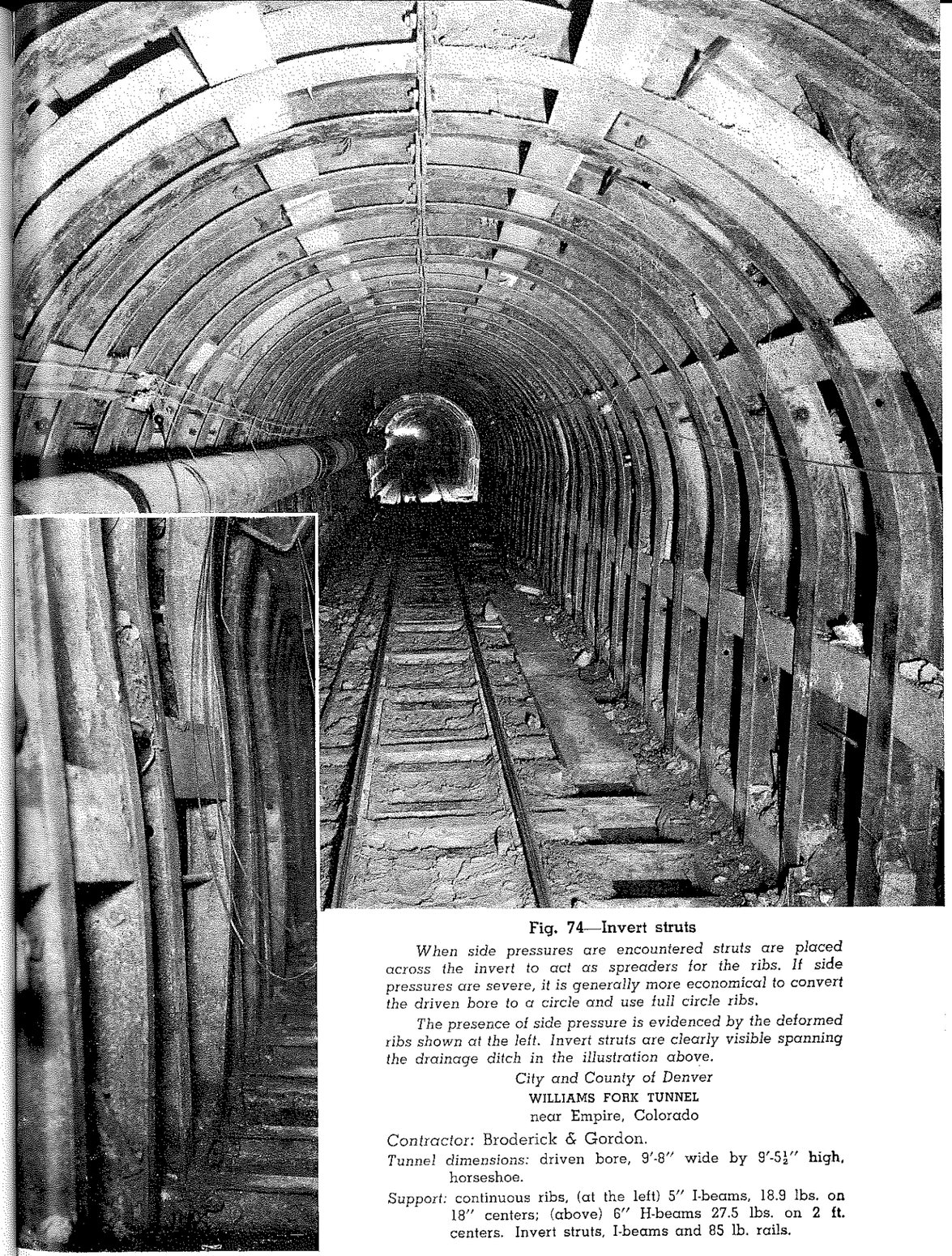


Fig. 74—Invert struts

When side pressures are encountered struts are placed across the invert to act as spreaders for the ribs. If side pressures are severe, it is generally more economical to convert the driven bore to a circle and use full circle ribs.

The presence of side pressure is evidenced by the deformed ribs shown at the left. Invert struts are clearly visible spanning the drainage ditch in the illustration above.

City and County of Denver
WILLIAMS FORK TUNNEL
near Empire, Colorado

Contractor: Broderick & Gordon.

Tunnel dimensions: driven bore, 9′-8″ wide by 9′-5½″ high, horseshoe.

Support: continuous ribs, (at the left) 5″ I-beams, 18.9 lbs. on 18″ centers; (above) 6″ H-beams 27.5 lbs. on 2 ft. centers. Invert struts, I-beams and 85 lb. rails.

Invert struts are used under these conditions. Such struts are shown in *Fig. 74.* These members extend across the tunnel at subgrade. They are so attached to the vertical members that they receive the horizontal pressures and in addition all or part of the vertical loads. They are curved to form an inverted arch. Curved struts can transmit to the invert much heavier loads than straight beams of equal weight and distribute them more uniformly.

LONGITUDINAL MEMBERS

Types and functions of wall plate

Three types of wall plates are commonly used. They are designated as *Double Beam, Single Beam* and *Flat* Wall Plates.

Double Beam Wall Plates comprise two I-beams side by side, as shown in *Fig. 75,* webs vertical, with about a 4-in. space between flanges to give access to the clamping bolt and admit concrete. The beams are spaced by vertical diaphragms welded under each rib seat. Ribs and posts are clamped by toggle plates and bolts, thus avoiding the time required for matching bolt holes. This method of attachment also permits variable spacing of either or both the ribs and the posts. This type of beam provides a broad surface of contact for blocking and to engage ribs and posts. Its box section makes it stable with respect to rolling and twisting.

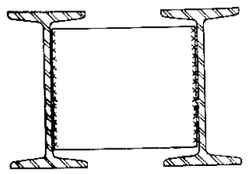


Fig. 75

Single Beam Wall Plates are H-beams, with web vertical as illustrated in *Fig. 76.* To enable them to transmit vertical loads from rib to post, they are reinforced at each rib seat with vertical T-shaped diaphragm plates. Attachment of ribs and posts is made by bolting through the flanges. This type of wall plate has been largely superseded by the "Double," Fig. 75, which permits faster rib and post erection and variable spacing, and is stronger and cheaper.

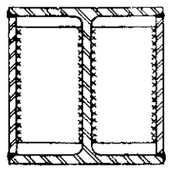


Fig. 76

Flat Wall Plates, Fig 77 are I-beams or Wide Flange beams used with their webs horizontal. They function merely as a cap for the posts and a sill for erecting roof ribs. The web is punched with vent holes to prevent trapping of air when concrete is poured. These holes also serve to pass reinforcing rods if any are required in the concrete design.

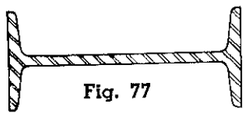


Fig. 77

The double and single beam wall plates are intended to resist bending in vertical planes. Therefore they can be and are used to transmit the loads from the ribs onto blocks or posts with a spacing different from that of the ribs. On the other hand, flat wall plates merely serve as an erection expedient and a convenient surface for horizontal blocking. Their resistance to bending in vertical planes is very small. Hence if flat wall plates are used, a post must be placed under each rib.

If the term "Wall Plate" is used in this text without qualification, it always indicates a wall plate with vertical web: namely, the double or single beam types. Wall plates with horizontal webs are specifically referred to as "flat" wall plates.

Position of wall plates in tunnel

The wall plate of the "Rib and Wall Plate" type of support is placed well outside the design concrete line in a hitch or pocket blasted out to receive it as shown in *Fig. 78.* The bottom is usually at, or some small distance above, the spring line.

When so placed it can be supported vertically by blocks or short posts which will be entirely outside the design concrete and therefore will not be unduly disturbed by the bench shots. These posts will vary in length and spacing to meet favorable spots in the overbreak. The posts may be of wood or steel, preferably of steel.

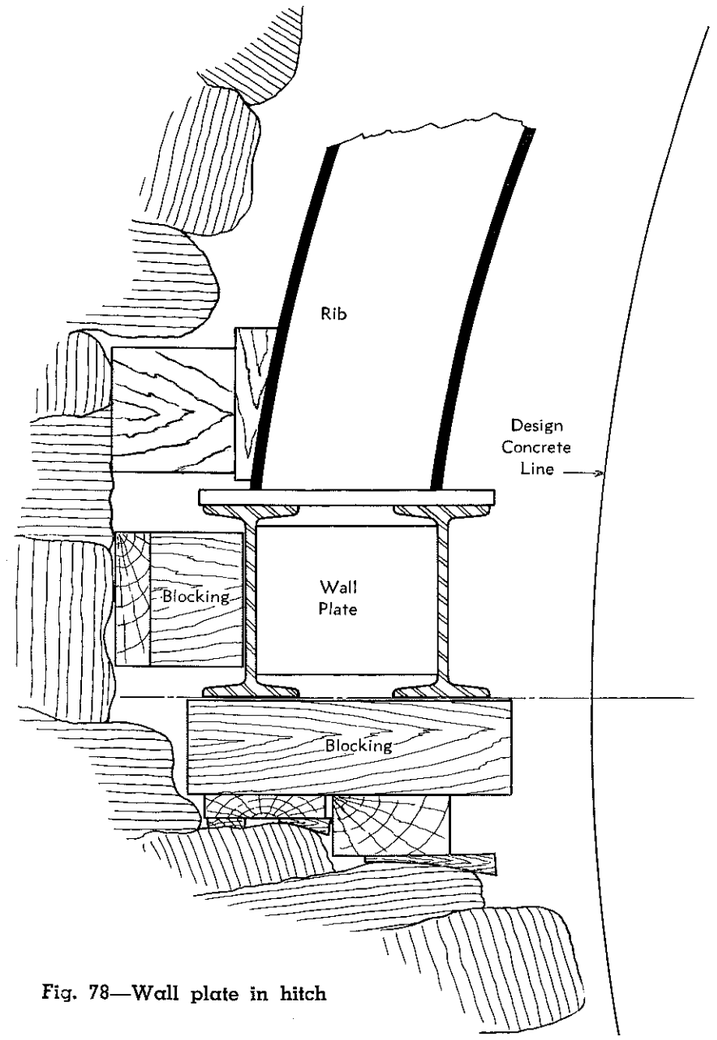


Fig. 78—Wall plate in hitch

On the other hand, as used in the "Rib, Wall Plate and Post" type of support, the wall plate is located wholly or partly within the design concrete line so as to minimize the width required to get in the posts which extend down to subgrade.

Wall plate in full face operation

When used in a full face operation, the wall plate is erected on pins projecting from holes drilled in the side wall just below the wall plate elevation. It is blocked to

line and grade, and then the roof ribs are erected and blocked. See *Fig. 62*, page 126. Posts and blocks are subsequently installed to support the wall plate vertically and horizontally, using favorable spots in the overbreak. Roof support erection may be carried tight to the face because of a short bridge-action period but if this period is long enough, support may be erected some distance back from the face.

Wall plate in heading

When the "Heading and Bench" or "Top Heading" method is used, it is because early support is needed for the roof. When this method is used the wall plate is placed on the bench, blocked to line and grade, and the roof ribs are immediately erected thereon. This use places a definite limit on the length of the wall plate, which cannot exceed the length of the top heading shot. It further limits the length of the bench shots, as these cannot exceed the length of the wall plate.

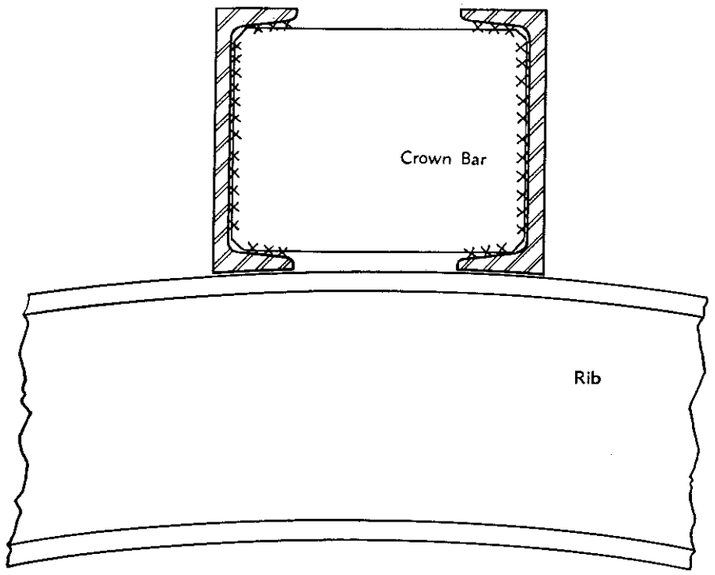


Fig. 79—Steel crown bar

In cases where the heading is knocked down with light shots, the wall plate would be entirely too short. Wall plate drifts are then shot out ahead of the heading face and the wall plates in appropriate lengths are blocked therein.

Blasting the bench takes out the blocking from under the wall plate for the length of the shot. The forward end rests on the remaining bench, but the rear end is carried by the splice attaching it to the next wall plate which, in turn, is carried by the posts. Therefore, its capacity is limited.

In most rock this support is sufficient to carry the roof until the posts can be installed, but in some cases rather severe loads come quickly. In such cases, supplementary support can be supplied by "Crown Bars" or "Truss Panels."

Crown bars

Crown bars may be built up of double channels as shown in *Fig. 79* or may be

plain H-beams or square timbers. They are located parallel to the axis of the tunnel. They either rest on the outer flanges of the ribs as in *Fig. 80* or they are attached to the ribs in hangers as in *Fig. 81*.

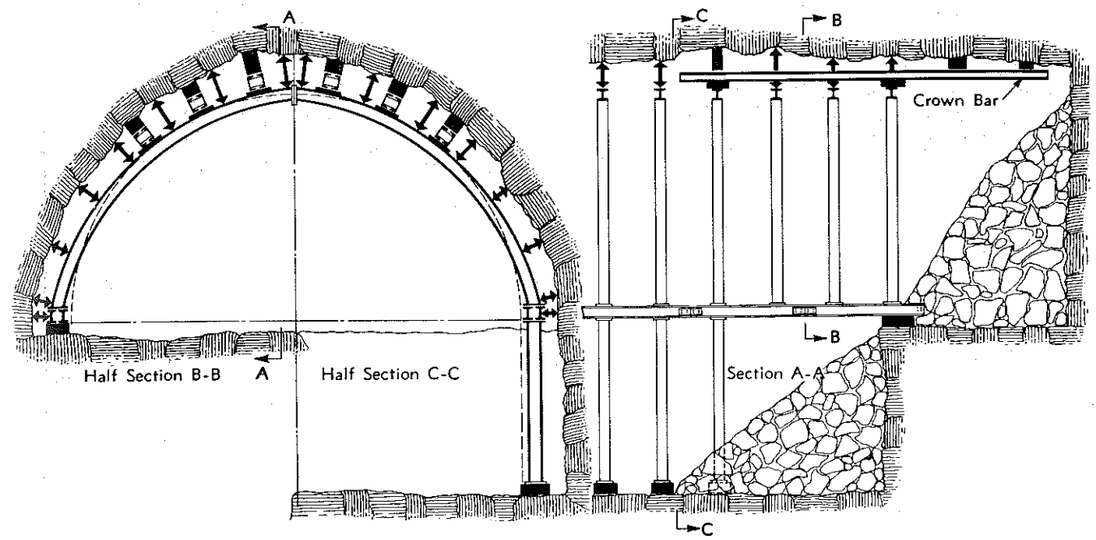


Fig. 80—Crown bars mounted on the ribs support the new roof by cantilever action

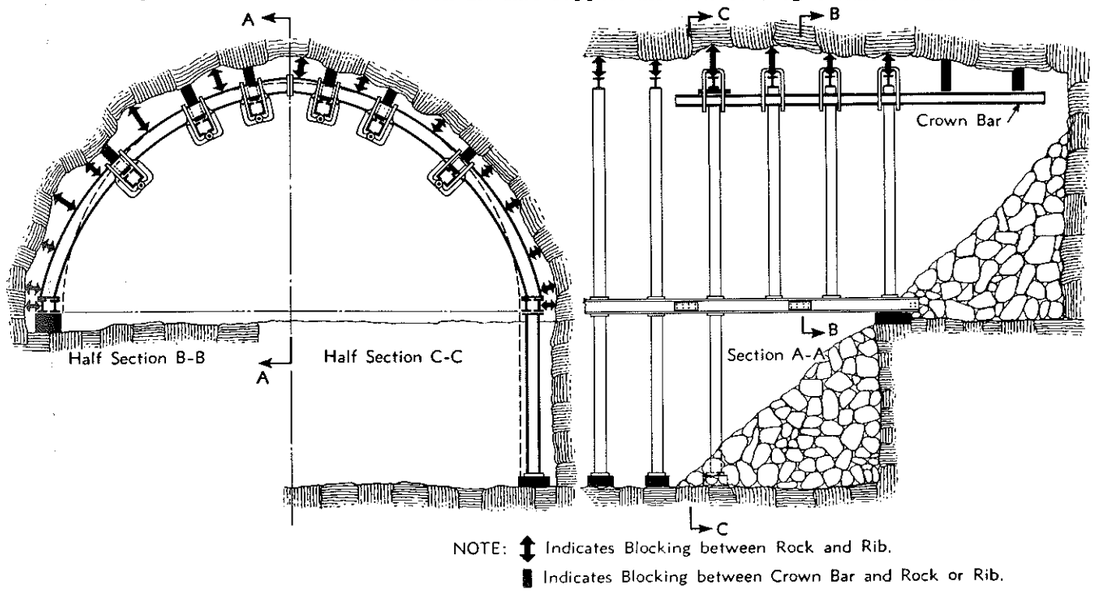
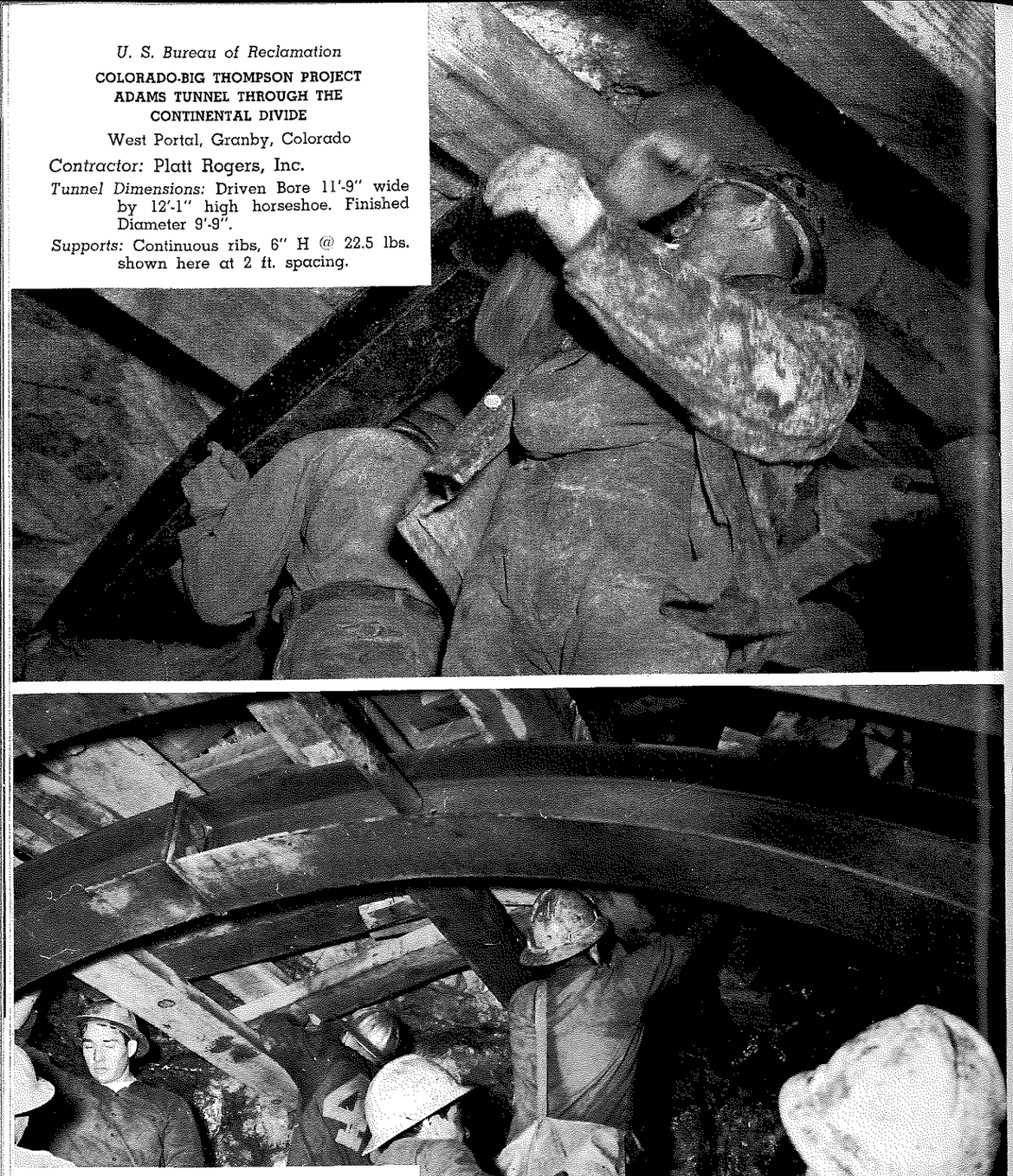


Fig. 81—Crown bars, hung from the ribs, support the new roof by cantilever action



U. S. Bureau of Reclamation

COLORADO-BIG THOMPSON PROJECT
ADAMS TUNNEL THROUGH THE CONTINENTAL DIVIDE

West Portal, Granby, Colorado

Contractor: Platt Rogers, Inc.

Tunnel Dimensions: Driven Bore 11'-9" wide by 12'-1" high horseshoe. Finished Diameter 9'-9".

Supports: Continuous ribs, 6" H @ 22.5 lbs. shown here at 2 ft. spacing.

Metropolitan Water District of Southern California

GLENDORA TUNNEL — FOOTHILL FEEDER

Contractor: John F. Shea Company.

Tunnel Dimensions: 19'-8" wide by 19'-4" high. Horseshoe Finished diameter 15'-6".

Support: Continuous ribs 8" H @ 34.3 lbs. shown here at 2 ft. spacing. In parts of the tunnel bottom struts were used.

Fig. 82—Crown Bars

Crown bars cantilever out from the last rib to give quick support to the roof. In Fig. 82 (b) men are inserting blocks between the roof and crown bars which have been slid forward under the bad top.

Fig. 82 (a) shows men erecting a rib under protection of crown bars which have just been slid forward and blocked.

Crown bars are an accessory, a construction expedient, intended to carry loads till the rib sets can be erected and the loads permanently transferred thereto. They have one of two functions: (1) to support the roof immediately after ventilating and thereby gain time for the installation of ribs, or (2) to support the roof or roof ribs over the bench shot thereby relieving or supplementing the wall plates.

It happens frequently that a short bridge-action period requiring the heading and bench method and crown bars for immediate roof support (function No. 1 above) is accompanied by heavy loading. This loading may exceed the ability of the wall plates to carry it across the bench shots. For this condition a second set of crown bars is required to supplement the wall plates, (function No. 2 above). It is not possible to assign both functions to one set of crown bars as they are needed at different stages of the excavating cycle. The crown bars that carry the new roof (function No. 1) must be free to move forward immediately after blasting, whereas the set that relieves the wall plates cannot be free to advance till mucking and posting under the wall plates is done, which will be some considerable time later.

Crown bars for immediate support are advanced to the face as soon after ventilating as the roof can be scaled down. When blocked, they support the roof by cantilever action from the last rib as shown in *Figs. 80* and *81*. *Fig. 82* shows crown bars supporting the roof while installation of the ribs proceeds. Crown bars are also shown in *Fig. 1 b*, page 18. If the mining operation has been conducted without crown bars, *emergency crown bars* of heavy timbers may be placed as shown in *Fig. 83*. Having thus caught up the roof time is gained to erect and block the roof ribs to relieve the crown bars, which are thus freed for the next advance.

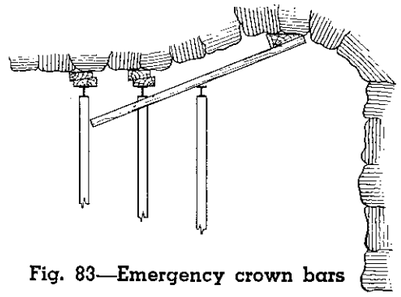


Fig. 83—Emergency crown bars

Crown bars to supplement wall plates are not advanced till mucking, posting, and erection of the new roof ribs in the heading has been completed. This is usually done while drilling for the next round. When advanced, the forward ends are supported by the newly erected ribs and the rear ends by the newly posted ribs, thus spanning the next bench shot. If the crown bars are located outside the ribs, that section of roof over the next bench shot is blocked to the crown bars as shown in *Fig. 84*. On the other hand, if the crown bars are attached to the inside of the ribs, the ribs over the bench shot are wedged from the crown bars as shown in *Fig. 85*. Thus they function to carry the roof loads for the next cycle.

When it is necessary to advance by short shots the crown bars relieve the load on the wall plates and thereby eliminate the necessity for wall plate drifts.

The manipulation of crown bars involves considerable extra work, performed at the most awkward place in the tunnel, in the crown. Furthermore, if the crown bars are positioned outside the ribs as in *Figs. 80* and *84* considerable extra excavation, concrete and grout are required. If immediate roof support is required the use of crown bars usually cannot be avoided. But when loads are too heavy for wall plates to sustain over the bench shots, another accessory has been devised which eliminates the crown bars and even the wall plates themselves. These are "Truss Panels."

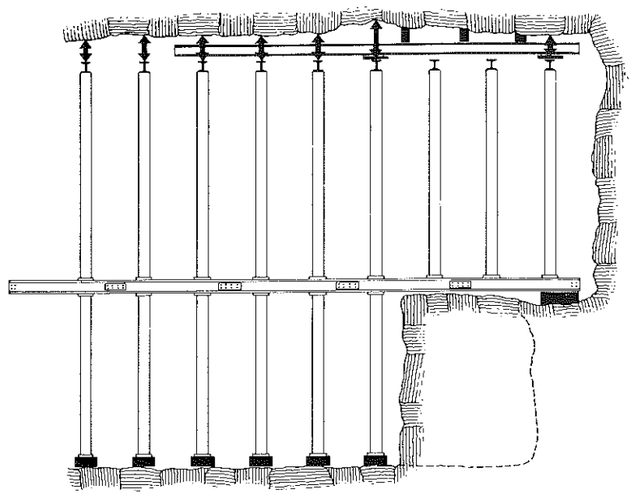


Fig. 84—Crown bars mounted on the ribs support the roof over the bench shot

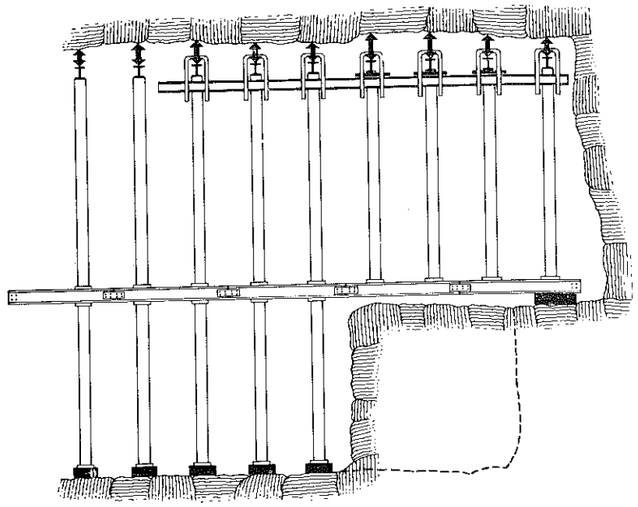
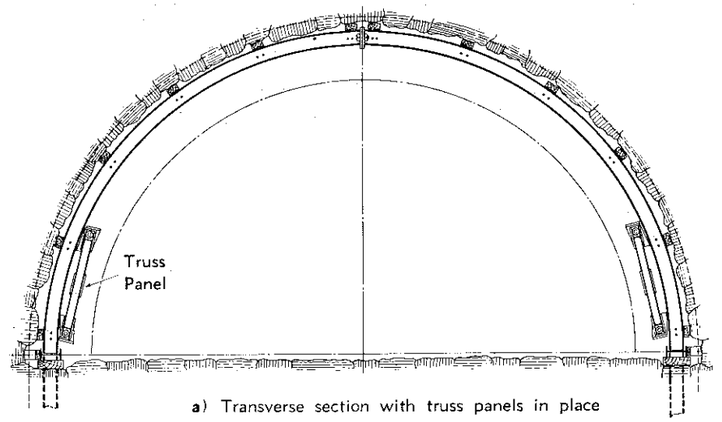


Fig. 85—Crown bars hung from the ribs support the ribs over the bench shot

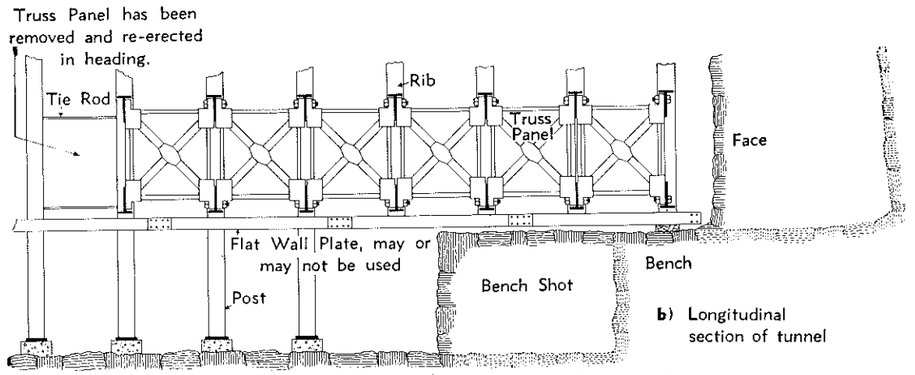
Truss panels

Truss Panels constitute an accessory for use with the combination of "Rib and Post" types of support, the "Heading and Bench" or "Top Heading" methods of attack, and heavy roof loads. Their purpose is to form, in combination with the ribs, a truss to span the gap produced by the bench shot.

The truss panels are attached to the inside face of the ribs for a distance of one or more ribs ahead of the bench shot, as shown in *Fig. 86* and are left there until posts are installed, at which time they are removed and sent up ahead. Attachment is by means of only two bolts at each rib. The truss thus formed may even be designed to carry the roof over two bench shots thus making it more convenient to get in the posts.



a) Transverse section with truss panels in place



b) Longitudinal section of tunnel

Fig. 86—Truss panels used to support roof ribs while bench is excavated

When truss panels are used, no wall plate is required although the flat wall plate may be used to keep the lower ends of the ribs lined up laterally if it is difficult to block the individual ribs against the rock. The truss panels also eliminate the need for wall plate drifts. The initial cost of a set of truss panels and the necessary lugs on the ribs is far less than the cost of continuous wall plates. The labor cost of moving truss panels is much less than the cost of handling crown bars.

Flat wall plates

A *Flat* wall plate is a "Wide-Flange" beam or an I-beam with its web horizontal and flanges vertical as already explained. It should be wide enough between the flanges to receive the ribs and posts. The major axis of the flat wall plate is horizontal, which permits of placing irregularly spaced blocks to take the horizontal thrust of the regularly spaced ribs above. Having comparatively little strength in the vertical direction, they require a post under each rib.

The flat wall plate is particularly useful under the following circumstances: (1) when using the "Rib, Wall Plate and Post" type of support with "Full Face," and "Side Drift" methods of attack, the posts are erected first, capped by the wall plate which forms a slot or groove in which to quickly set the roof ribs, and provides a surface against which the horizontal blocking can be wedged; (2) when using the same type of support in the "Heading and Bench" or the "Top Heading" attacks, with "Truss Panels" to bridge the bench shot, the flat wall plate may be used to keep the rib feet lined up horizontally and to take the horizontal thrust from the ribs; (3) when using the "Full Circle Rib" type of support, with earth tunneling methods through crushed rock in a fault zone or elsewhere, the flat wall plate may be used to advantage. The ribs are divided at spring line and the tunnel driven heading and bench without or with wall plate drifts, as shown in *Fig. 66,* page 131.

Bracing

Longitudinal bracing serves to increase the resistance of ribs and posts to buckling about their minor axis and to prevent a displacement of these set members during blasting. If the space between the ribs or posts is bridged by lagging which is firmly attached to the webs, no such bracing is required.

The most common types of bracing are known as "Tie Rods and Collar Braces" *Figs. 87* and *88,* and "Spreaders" *Fig. 89.*

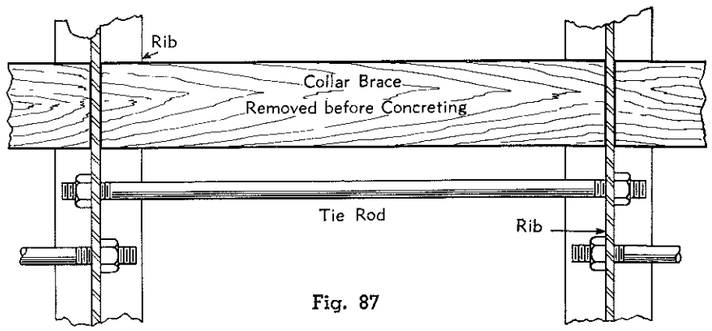


Fig. 87

Tie Rods usually are 5 8-in. or 3/4-in. rods, with thread and a nut on each end. The length is slightly more than the spacing of the ribs. *Collar Braces* are commonly pieces of timber, 3 x 4-in., 4 x 6-in., 6 x 6-in., or any convenient size. The length is equal to the rib spacing minus the web thickness. Holes in pairs, are provided in the web of ribs and posts for the tie rods. Collar braces are set in line between ribs, tie rods entered and the nuts tightened. Wood collar braces are usually removed before concreting.

Spreaders may be angles, channels, or I-beams, with a clip angle or plate welded on each end for bolting to the rib sets. These are left in the concrete.

In tunnels where rib spacing varies, the tie rod and collar brace type of bracing can be conveniently employed, as the tie rods can be furnished in "mill" lengths, to be cut off and threaded on the job. They and the collar braces can be cut to length as required by each day's mining.

Spacing of bracing is usually determined so that there will be a brace near the end of each member of the rib set, with intermediate braces spaced not over 5 ft. apart.

Fig. 88—Tie Rod and Collar Brace bracing

Typical use of "Tie Rod and Collar Brace" type of bracing. The wood collar braces prevent displacement of the rib by falling muck. They usually are removed at some distance back from the face and reused ahead. Generally all wooden collar braces are removed before concreting.

U. S. Bureau of Reclamation

YAKIMA PROJECT, YAKIMA RIDGE CANAL, ROZA DIVISION

Tunnel No. 3 near Yakima, Washington

Contractor: Morrison Knudsen Company

Tunnel dimensions: driven bore, 19'-1" by 19'-1", horseshoe; finished bore, 16'-9" by 16'-9", horseshoe.

Support: continuous ribs, 6" I-beams, 12.5 lbs. spaced at 4'-0" centers.

LAGGING AND PACKING

Function of lagging

The term *lagging* is applied to those members of a tunnel support which span the spaces between the main supporting ribs. The elements which compose the lagging are called *lags.* Lagging has one or more of the following functions to perform:

To provide protection from falling rock or spalls.

To receive and transfer loads to the rib sets.

To provide a convenient surface against which to block in case it is not convenient to block directly against the rib, because of irregular overbreak.

To provide a surface against which to place backpacking.

To serve as an outside form for concrete lining if concrete is not to be poured against the rock.

To divert water, and so prevent leaching and honeycombing of concrete.

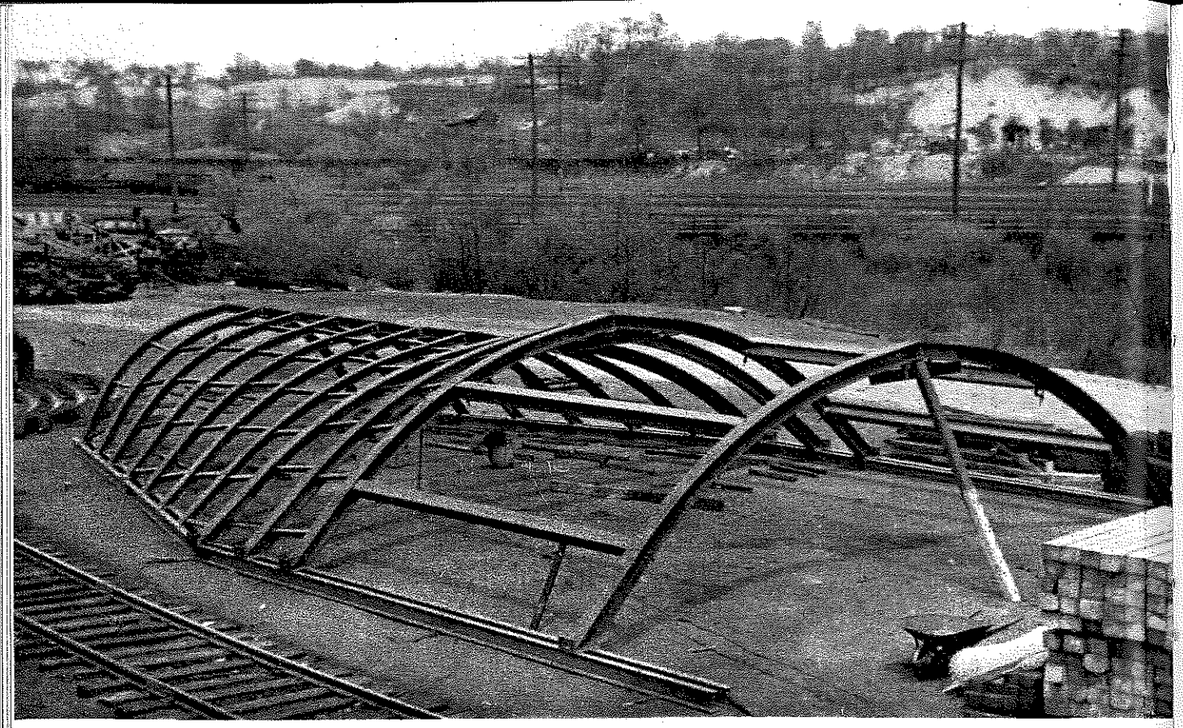


Fig. 89—Spreaders.

Spreaders are permanently bolted between the webs of the ribs and are concreted in. Unless spreaders are procured in different lengths, the rib spacing is fixed. In this photograph, showing the steel support for the enlargement at the bottom of a shaft, the spreaders were I-beams, although channels and angles are frequently used for this purpose.

Board of Water Supply, New York City
DELAWARE AQUEDUCT, ROUNDOUT-WEST BRANCH TUNNEL
near Newburg, New York

Contractor: Walsh Construction Co.

Tunnel dimensions: driven diameter, 18′-5″; enlargement, 36′-6″ out to out of wall plates; finished diameter 13′-6″.

Support: rib and wall plate type; ribs at shaft 8″ x 8″ WF-beams, 40-lbs.; elsewhere in the enlargement 6″ H-beams, 27.5 lbs.; wall plates, double 8″ I-beams.

Lagging: 6″ channels, clamped to ribs.

Types of lagging

Lagging can be made either of wood or steel. The different types of lagging are commonly given the name of the kind of units which compose the lagging as indicated in *Fig. 90.*

Wood Lagging, Fig. 90 a, consists of hardwood planks, 3 in. or 4 in. thick by 6 in., 8 in. or 10 in. wide, which are cut to lengths slightly less than the rib spacing. These are placed usually on the outside flange, occasionally on the inside flange of the ribs, and may be spaced or tight.

Channel Lagging, Fig. 90 b, consists of rolled steel channels, 4, 5, or 6 in. wide, which are placed on the ribs flatwise with their flanges outward. Pressed steel channels of various dimensions may be used. This is the steel equivalent of wood lagging.

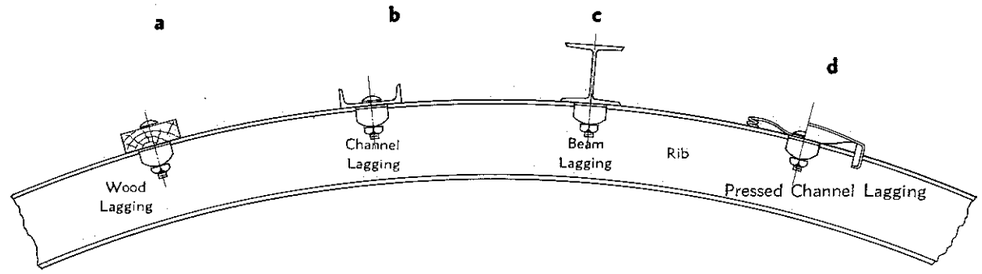


Fig. 90—Types of lagging

Beam Lagging, Fig. 90 c, consists of rolled steel H-beams, 4 or 5 in. deep which are attached to either flange of the rib and spaced at suitable distances apart.

Pressed Channel Lagging, Fig. 90 d, consists of channels pressed from plate or strip, having return flanges to develope high strength. Ends are reduced in depth so minimum head room is required. Commonly 12 in. wide and 2 in. deep of thickness to suit conditions.

Liner Plate Lagging, Fig. 91, consists of the pressed steel liner plates commonly used in earth tunneling, and are installed between the webs of the ribs to make a tight lagging. This is ideal for some purposes. They may also be bolted together outside the ribs to make a continuous steel skin which is supported by the ribs through wedges.

Fig. 91—Steel Liner Plate used extensively as lagging in earth tunnels and ideal for lagging in many rock tunnels

Purlin-Plate Lagging, Fig. 92, consists of light steel plates, plain or corrugated, which are fastened to purlins made of angles, channels or I-beams. The purlins are installed between the webs of the ribs. The plates attached to the purlins are wide enough to cover the entire space between the outer flanges of the ribs, thus forming a tight skin.

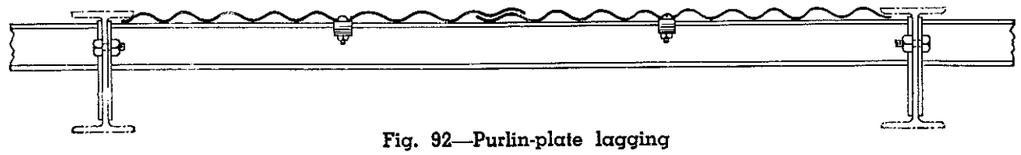


Fig. 92—Purlin-plate lagging

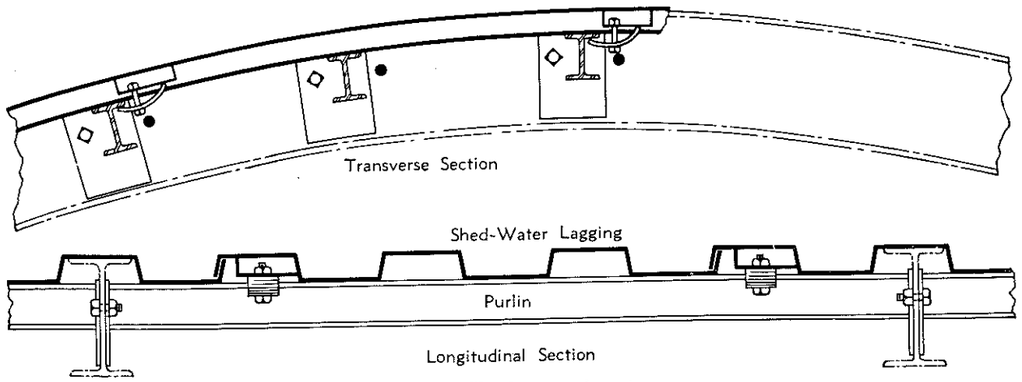


Fig. 93—Shed-water lagging

Shed-Water Lagging, Fig. 93, is a purlin-plate type of lagging in which specially ribbed light plates cover the outer flanges of the rib sets as well as the purlins. Adjacent plates overlap in both directions, thus enclosing the whole structure in a water-diverting skin of steel. All fastenings are welded to the inside to avoid the possibility of leakage through the lagging. This lagging diverts dripping or flowing water to the sides of the tunnel where it may be drained off.

Spacing between lags

The lags which compose certain types of lagging can be placed either with fairly wide interstices between them or else skin tight. Lagging with spaced lags is known as *"Skeleton"* or *"Open"* lagging in contrast to the *"Tight"* lagging which constitutes a continuous, rib-supported shell.

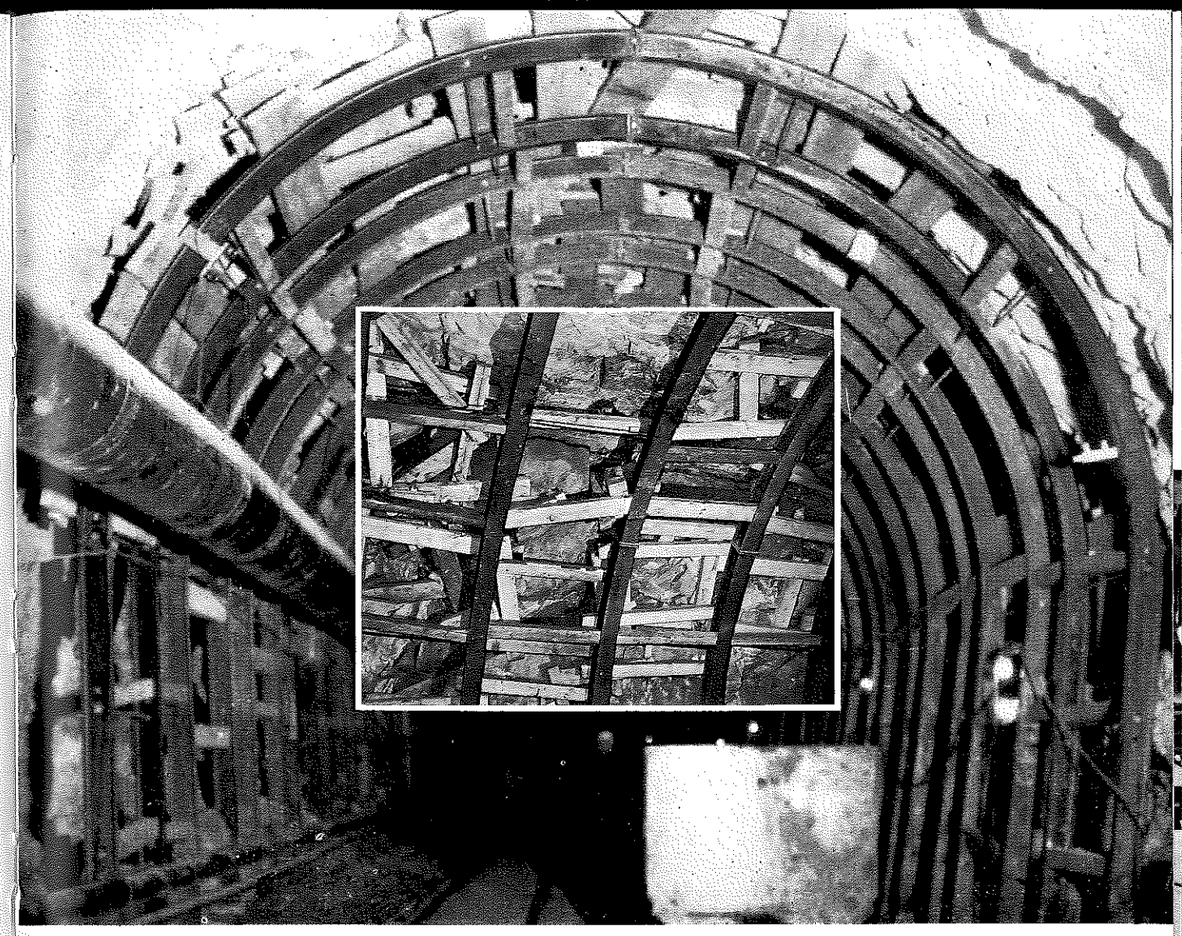
Skeleton lagging

The skeleton lagging is intended to receive loads from rock located between ribs without preventing the flow of concrete into the space behind the lags. By far the greatest footage of tunnel is driven with this form of lagging, suitable as protection against spalls, and to hold stratified broken rock. *Fig. 94* illustrates this type. The lags are spaced far enough apart to permit passage of the concrete (not less than 3 in.), but yet close enough to catch any dangerously large spall. When the job is ready for concreting, lags which are not under load are removed to allow the easiest possible passage of concrete.

Wood, Channel, or Beam lagging are the types used for skeleton lagging. Anticipated loads, space limitations, rib spacing, and comparative cost determine the choice. As a rule, the spacing will be closest at the crown, increasing rapidly down to spring line. On the sides only an occasional lag is used, if any. As there are inevitably a large number left embedded in the concrete, the current practice increasingly favors steel lagging. Wood may rot out in time creating semi-voids in the concrete; whereas steel, encased in concrete, is permanent.

Attachment of skeleton lagging

The elements of a skeleton lagging are fastened to the ribs by means of *bolts, clip angles* or *clamps.*



Inset is from Floyd Hill Highway Tunnel West of Denver
Contractor: Colorado Constructors, Inc.

Fig. 94—Skeleton lagging

Wherever it is desired to pour concrete against the rock, skeleton lagging is used if rock conditions permit. Most tunnels are lagged in this manner. Good tunnel practice requires that all wood lagging and blocking that is not actually under load be removed prior to concreting.

Metropolitan Water District of Southern California

LOS ANGELES-COLORADO RIVER AQUEDUCT, WEST IRON MOUNTAIN TUNNEL
near Rice, California

Contractor: Utah Construction Co.

Tunnel dimensions: driven bore, 18′-2″ wide by 17′-0″ high, horseshoe; finished bore 16′-0″ by 16′-0″ horseshoe.

Support: continuous ribs, 6″ I-beam, 12.5 lbs., spacing variable.

Bolt connection requires holes in both the rib flanges and the lags. Therefore the spacing between the lags is fixed, which is a disadvantage, as rock sometimes projects within the lagging line. Holes also deduct from the sectional area of the rib. The first cost is rather high and erection slow.

Clip Angles are either bolted or welded to the ribs. The clip angles locate the lags when laid on the ribs. The clip angle connections have the same drawbacks as the

bolted connections. To these must be added the danger that the lags may be jarred out of their position during blasting.

Clamp connections, Fig. 95, permit attachment of the lags at any desired spacing. Each lag is provided at each end with a bolt and a clamp so that it can be installed at any place. The nuts are attached to special bolts which cannot turn. By tightening the nut, the lag is firmly held in its position. The clamp which serves this function is referred to as a *Lagging Clamp.* The lagging clamp attachment is cheaper than any other but the important advantage is that the spacing can be varied from lag to lag. It can be applied to steel or wood lags. *Fig. 96* shows a tunnel with clamped skeleton lagging.

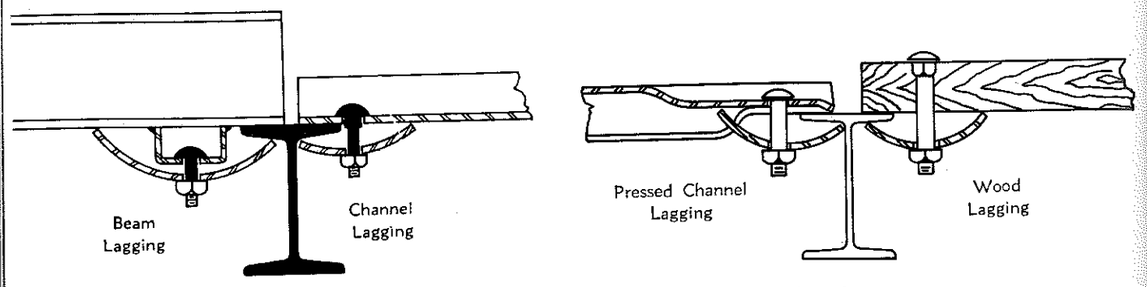


Fig. 95—Lagging clamp applied to various types of lagging

Tight lagging

Tight lagging is employed where rock conditions make it necessary to backpack the lining or where a considerable quantity of water flows through the roof of the tunnel. Such rock conditions may occur in decomposed rock, crushed rock in fault zones, extremely broken rock, squeezing and swelling rock, popping rock and also in certain kinds of shale. *Figs. 97* and *98* show two types of tight lagging, wood and liner plate.

Any tight lagging is sufficient to support backpacking and to prevent rock from moving into the bore. It can be made of any kind of lags by simply placing the lags tight together. But a special water-diverting or shed-water type of lagging is required where a heavy flow of water through the roof has not been stopped by grouting or other means. If this water reaches the fresh concrete of the lining it will wash out the cement, with consequent honeycombing and weakening of the structure. The elements of such shed-water lagging have been described above, and the lagging is usually placed at or beyond the design line of concrete, to divert the water toward the spring line.

Current practice has been to divert water by erecting a sheet metal skin within the rib sets before concreting as shown in *Figs. 99* and *100.* This "panning" is expensive and does not permit the ribs to be encased in concrete.

To overcome these disadvantages a shed-water type of steel lagging has been devised which encases the support structure like a steel skin. This lagging is laid over the ribs and purlins as soon as they are erected. Wherever the lagging overlies a rib, it makes metal to metal contact with the rib outer flange as shown in *Fig. 93,* page 158, thereby providing a firm support for the blocks. The space between the lagging and the rock is filled with a dry packing of rock fragments to act as a drain, as noted below.

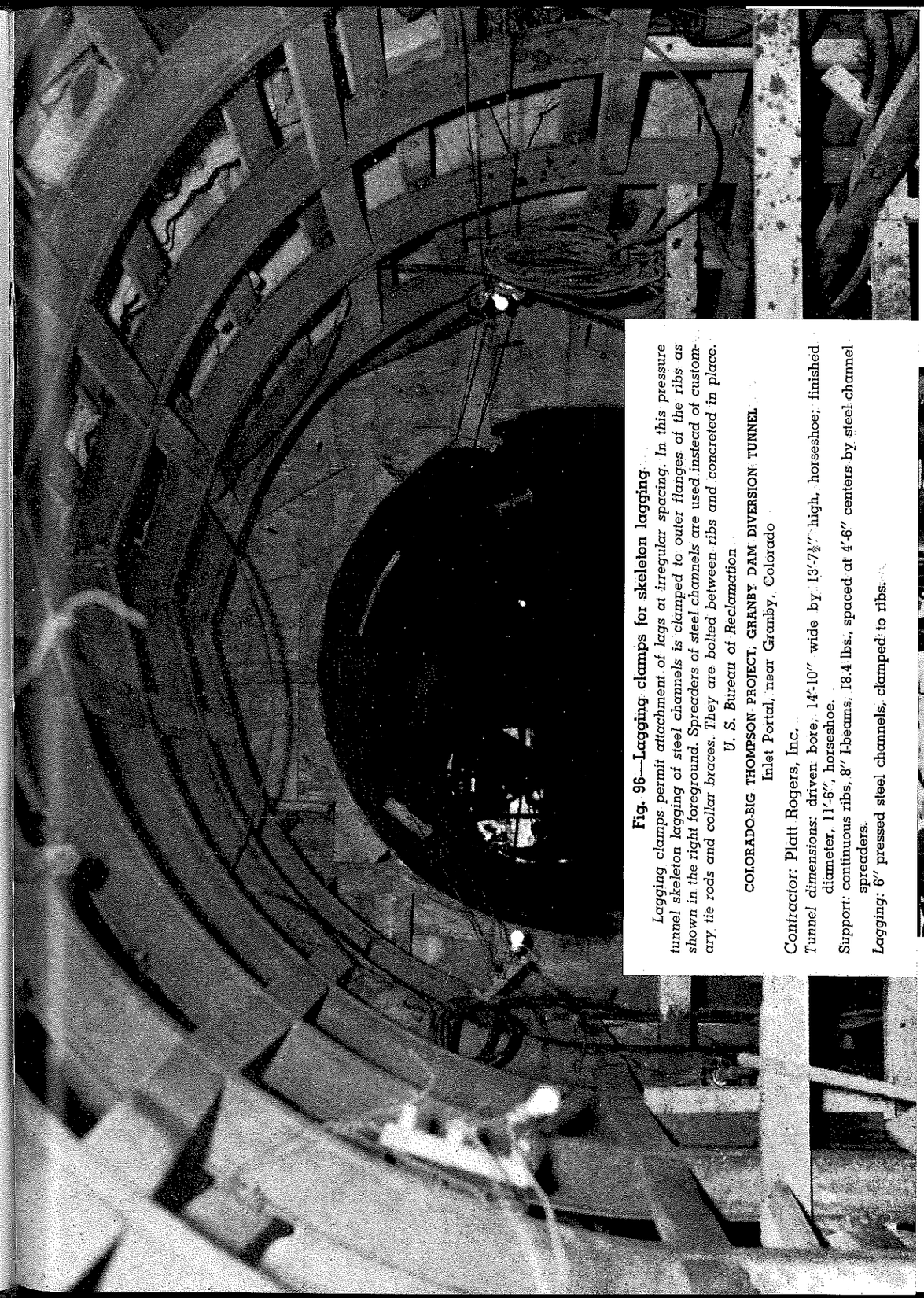


Fig. 96—Lagging clamps for skeleton lagging

Lagging clamps permit attachment of lags at irregular spacing. In this pressure tunnel skeleton lagging of steel channels is clamped to outer flanges of the ribs as shown in the right foreground. Spreaders of steel channels are used instead of customary tie rods and collar braces. They are bolted between ribs and concreted in place.

U. S. Bureau of Reclamation

COLORADO-BIG THOMPSON PROJECT, GRANBY DAM DIVERSION TUNNEL

Inlet Portal, near Granby, Colorado

Contractor: Platt Rogers, Inc.

Tunnel dimensions: driven bore, 14'-10" wide by 13'-7½" high, horseshoe; finished diameter, 11'-6", horseshoe.

Support: continuous ribs, 8" I-beams, 18.4 lbs., spaced at 4'-6" centers by steel channel spreaders.

Lagging: 6" pressed steel channels, clamped to ribs.

Fig. 97—Tight lagging of wood lags.

Wood lags placed tight to support dry pack.

U. S. Bureau of Reclamation

KENDRICK PROJECT, CASPER CANAL

Tunnel No. 5, near Casper, Wyoming

Contractor: W. E. Callahan and Gunther & Shirley.

Tunnel dimensions: driven bore, 16'-0" by 16'-0", horseshoe; finished bore, 14'-0" x 14'-0", horseshoe.

Support: continuous ribs, 5" I-beams, 12.5 lbs. on 3 to 5 ft. centers.

PINEY BRANCH TRUNK SEWER

Section 3, Washington, D. C.

Contractor: Leo Butler Company.

Tunnel Dimensions: 22'-6" wide by 19'-4" high, finished diameter 14'-6".

Supports: 6" H @ 20# ribs 2'-8" spacing generally 8" I @ 18.4# double beam wall plates.

Lagging: 16" x 37 11/16" liner plates.

Fig. 98—Tight lagging of steel liner plates.

Liner plate lagging installed outside of steel ribs for support of dry pack in tunnel roof. Ribs shown are 32" spacing which is every other course of 16" liner plates. Spacing of ribs may vary as necessary to support ground loads and if necessary additional ribs may be added. Note blocking between liner plates and back of ribs. The shaft shown is supported in a similar manner.

Below the spring line, two different methods can be used for protecting the concrete. One method is to carry the shed-water lagging and the dry pack down to subgrade and to collect the water in drains located at subgrade. The second method is to install on the sides of the tunnel a system of blind drains sealed to the rock wall. The water which comes from the roof is diverted into these drains at the spring line. Below the spring line the drain system consists of vertical drains and of laterals which collect the

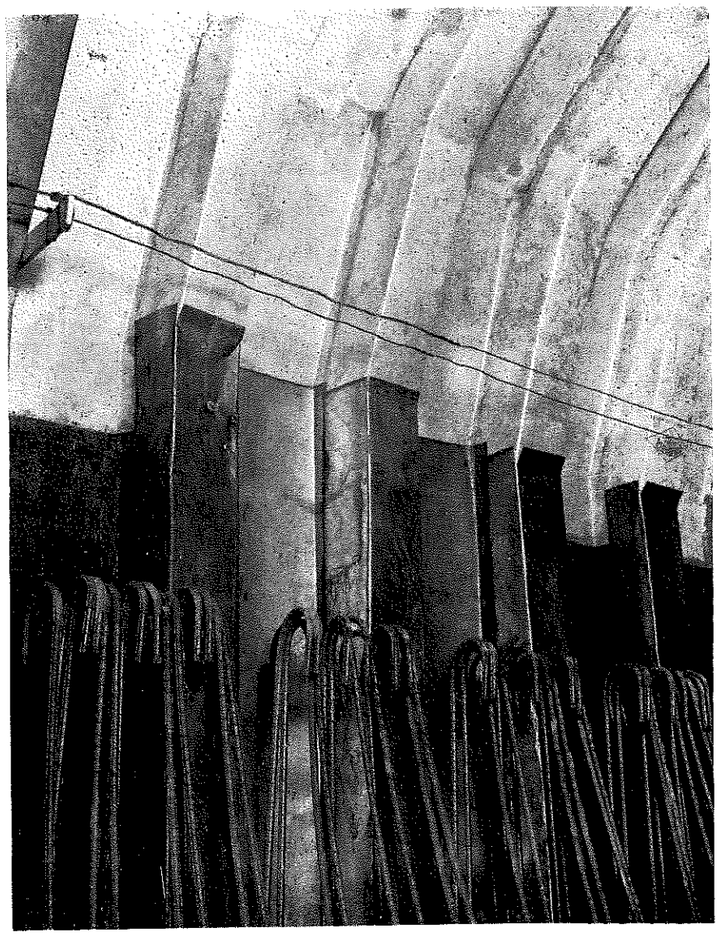


Fig. 99—Protection of concrete from water.

Sheet metal panning, installed to divert water entering tunnel roof and side walls, entirely encloses the timber sets. Water is diverted behind the panning to drains at subgrade.

Metropolitan Water District of Southern California
LOS ANGELES-COLORADO RIVER AQUEDUCT, VALVERDE TUNNEL NO. 3
near Valverde, California

Contractor: Dravo Contracting Co.

Tunnel dimensions: driven bore, 19′-0″ wide by 18′-8″ high, horeshoe; finished bore, 16′-0″ by 16′-0″, horseshoe.

water coming out of seams located between the vertical drains. If the second method is used, the concrete above the spring line is poured against the shed-water lagging, whereas below the spring line it is poured against the rock.

Rock conditions requiring packing

The term *packing* indicates the material which is used to fill the empty space between the lagging and the rock surface. The function and type of packing depends on the rock condition. In dry tunnels through jointed rock, packing is only used to fill large cavities produced by excessive overbreak. In broken, crushed or decomposed

Board of Water Supply, New York City
CITY WATER TUNNEL NO. 2
New York City

Contractor: Patrick McGovern, Inc.

Tunnel dimensions: driven diameter, 17′-9″; finished diameter 15′-0″.

Panning: ribs, 2—5″ x 3½″ x ½″ angles back to back at 4 ft. centers; lagging 1/8″ x 24″ x 96″ plates.

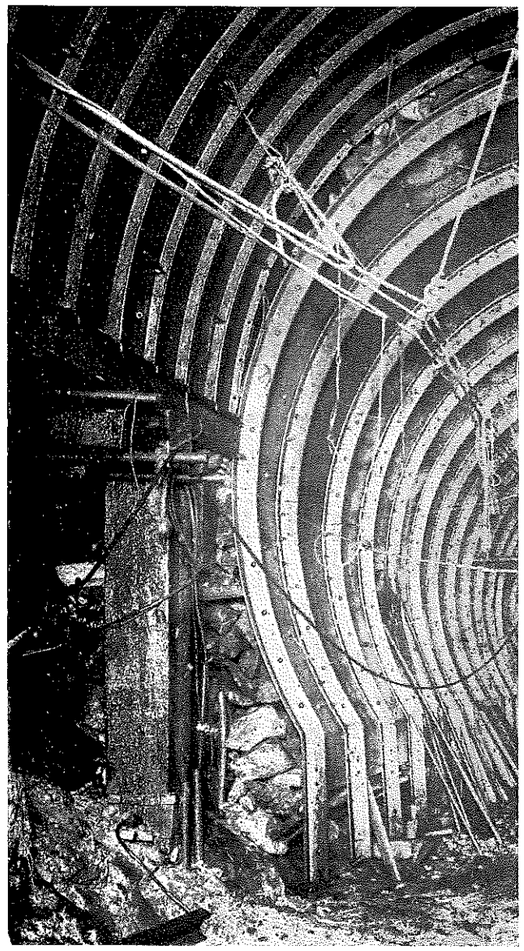


Fig. 100—Panning to divert water.

To divert water while placing concrete the panning structure shown in this illustration was erected below the roof ribs in wet stretches. Dry pack was installed behind the panning to provide drainage to subgrade. The shed-water type of lagging has been developed to avoid the expense of such structures.

rock it serves to transfer the rock load onto the lags, thereby acting as a substitute for excessive blocking. In squeezing rock it provides continuous contact through the lagging with the rib sets. In jointed, water-bearing rock it has primarily the function of a drain.

The type of packing depends on the purpose to be served and, at equal suitability, on the cost of labor and material.

Packing in large cavities

Large cavities can be present in the rock prior to tunneling, such as caves in limestone. They may originate during mining, by masses of rock dropping out of the roof before the support is installed, or by broken or crushed rock flowing into the tunnel through gaps between lags.

If the cavities are not wider or higher than a few feet, the communication between cavity and the tunnel is intercepted by tight lagging and the cavity is filled with "one-man stones" retained by the lagging. The stones eliminate the necessity to prop up the roof of the cavity and they reduce the quantity of grout which is required to seal the hole after the permanent lining is built.

If the height of a cavity above the roof exceeds ten or fifteen feet, it is customary to cover the lagging beneath the hole with tunnel spoil to a thickness of five to ten feet and to leave the balance of the cavity open. The layer of spoil cushions the impact of stones which may drop out later from the roof of the cavity.

Packing in seamy, water-bearing rock

In seamy, water-bearing rock the tunnel support is lined with shed-water lagging and the packing serves as a drain. Such dry pack can be made out of any accumulation of fairly sound pieces of rock, provided it does not contain any fragments smaller than two or three inches. If the rock in the tunnel is suitable, the dry pack can be obtained by hand picking or by passing the tunnel spoil through a screen.

Packing in broken, crushed, decomposed or slaking rock

When tunneling through the rocks listed in this heading the packing serves to transmit the rock load onto the lagging over the entire outside of the lagging. This purpose can be achieved by filling the space between lagging and rock with any suitable material such as tunnel spoil, dry or wet concrete, pea gravel or wood.

All these packing materials, with the possible exceptions of some types of wood packing, require tight lagging. If wood packing consists of poles or slabs, it can be used in combination with skeleton lagging or even without any lagging. However, wood packing is now largely discontinued, because the decay of the timber gradually deprives the adjoining rock of its support and may lead to movements in the rock associated with the development of unsymmetrical loads. Many serious tunnel defects which developed years or decades after construction were found to be due to the decay of large quantities of timber left in the ground. After the tunnel lining is constructed, the material which occupies the space behind the lagging is consolidated by cement grouting.

Dry pack

Dry pack usually consists of tunnel spoil shoveled or hand packed into the space between the lagging and the rock. It is usually placed simultaneously with the erection of the lagging. Starting at the lowest point, a few lags are placed, and tunnel spoil is shoveled in behind. This procedure is carried up to the crown at which point it is necessary to pack endwise.

Dry pack, as shown in *Fig. 101,* may be used wherever packing is required but it must be used where water is to be diverted by the shed-water type of lagging. Any type of tight lagging may be used with dry pack. With liner plate lagging it is

Fig. 101—Dry packing

Dry pack placed over liner plate arch on concrete side walls in mine haulage way.
H. C. Frick Coal & Coke Co.
PALMER DOCK MINE, BELT CONVEYOR TUNNEL
near Connellsville, Pa.

Contractor: Force Account
Tunnel dimensions: span of liner plate arch, 14′-6″; radius of liner plate arch, 8′-4″.
Liner Plates: 3/16″ x 16″ x 36″.

necessary to pack lengthwise of the tunnel, course by course, instead of proceeding as indicated above.

Concrete packing

Concrete packing is used for the same purposes as the dry pack. It may be more expensive but, on the other hand, it consolidates without being grouted. The fact that it consolidates and becomes rigid excludes the concrete packing from use in connection with tunnel supports which are expected to yield. (See *Chapter 8*). In all other

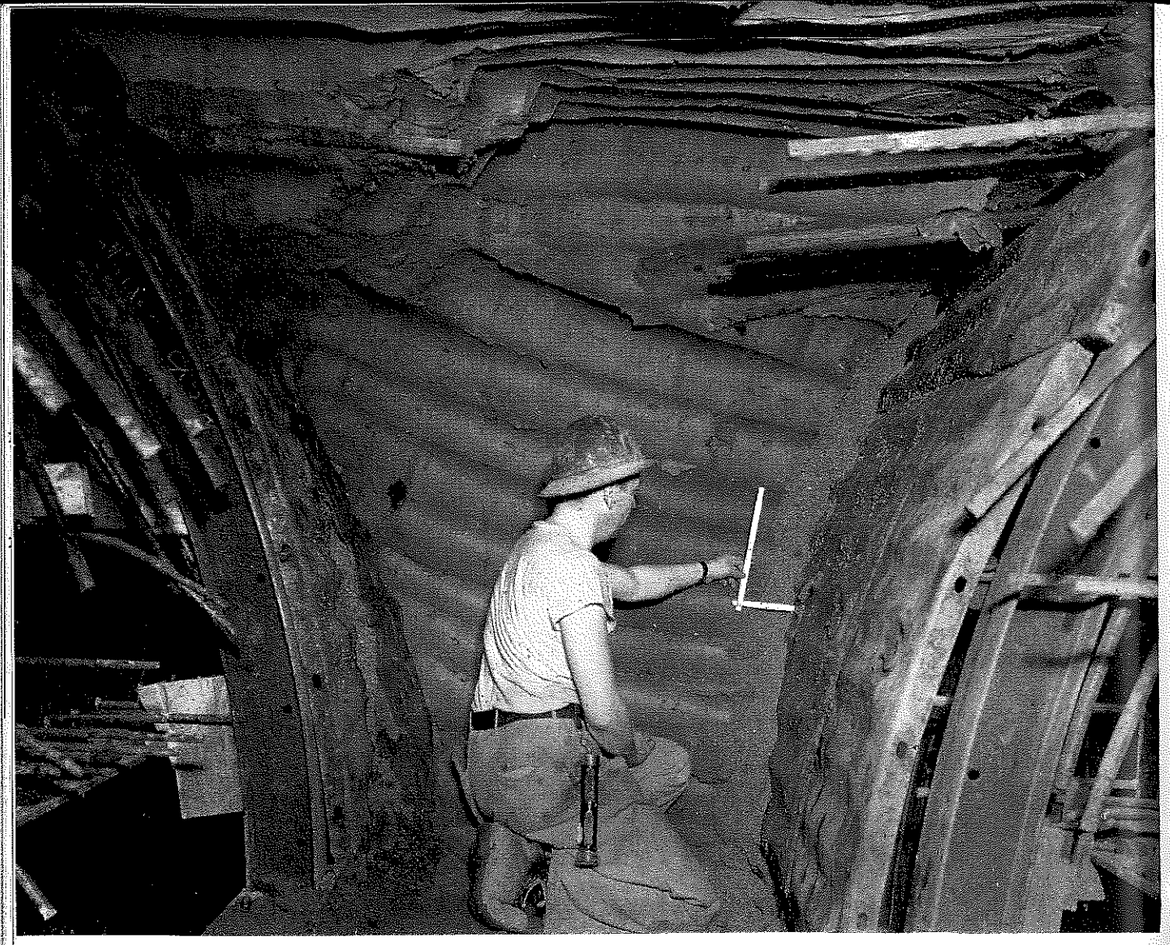


Fig. 102—Gravel packing

Pea gravel packing blown through liner plates was used extensively in the Chicago Subway to fill space left by over-mining. This view shows gravel about 4-in. thick at right side and in irregular pockets at left side. Grout holes in two liner plates, closed with grout hole closers, are shown on line with the rule.

City of Chicago, Dept. of Subways & Superhighways
CHICAGO SUBWAY, DEARBORN ST. LINE
Section D4, under Milwaukee Avenue

Contractor: John Marsch, Inc.

rocks, excluding those with a moderate swelling capacity, the rigidity of the concrete packing is an advantage. Concrete packing is also excluded from use when shed-water lagging is used, as it does not provide drainage.

Dry Concrete Packing is ordinarily a very lean mix of concrete, 1-3-6 or 1-4-8. It is placed fairly dry in the same manner as dry pack. The disadvantage of dry pack and dry concrete is the large amount of hand labor involved in placing it. Also, the cycle time for erecting the support is markedly increased, which adds to the cycle time for the whole excavating operation.

Wet Concrete Packing should be of the leanest mix which can be placed with a pneumatic placer or concrete pump. If wet concrete is placed by machine, the support system can be erected and lightly blocked, allowing the other tunnel operations, such as drilling to proceed. The concrete can be placed while drilling is in progress.

Wet concrete packing can be used under the same circumstances as dry concrete. Liner plate lagging is the type most suitable for wet concrete, as it is tight enough to hold it.

Pea gravel packing

Pea gravel packing is a highly desirable kind of packing for most purposes. However, so far, it has not been practiced in rock tunnels. The gravel is blown through holes provided in the lagging or endwise over the last rib set. Since the cost of labor and material are low, this type of packing deserves the consideration of the rock tunnel builder.

Blowing of gravel is standard practice in shield-driven tunnels, to fill the annular space around the lining left by the advancing tail of the shield. Gravel is blown through the grout holes provided in the liner segments as the shield is shoved forward. Gravel

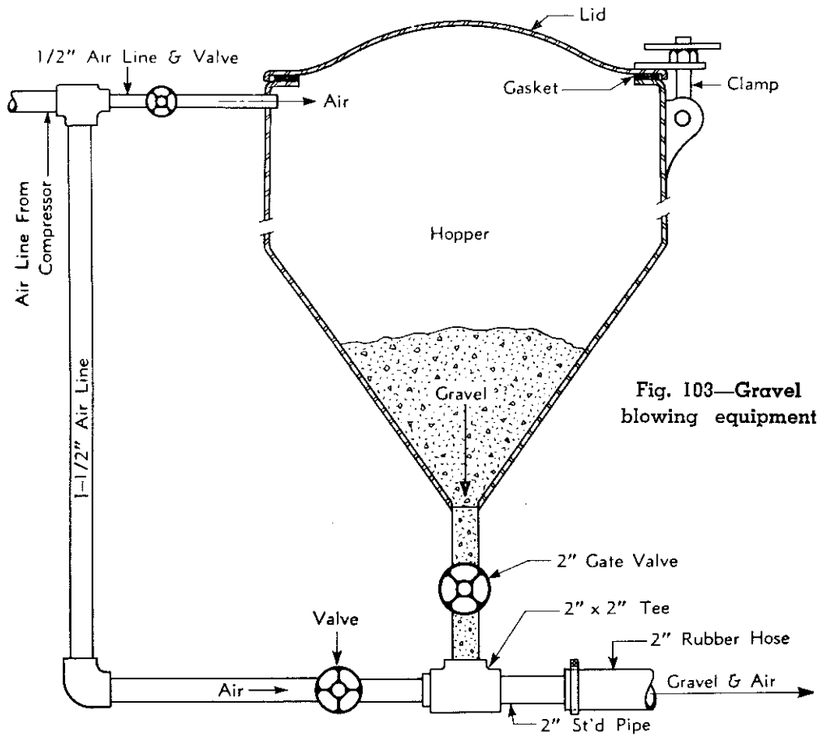


Fig. 103—Gravel blowing equipment

was used extensively in the Chicago Subway tunnels, not only on the shield-driven contracts but in the hand-mined tunnels to fill the overmined space outside the liner plates, as shown in *Fig. 102.*

The equipment for blowing gravel is quite simple. The essential parts are shown in *Fig. 103.*

The gravel hose usually terminates in a pipe nipple which can be shoved through holes in the lagging or backward over the last rib. Advantages of using gravel as backpacking are:

It permits the steel support to be quickly erected. Dry pack or dry concrete slows down support erection as these packings are installed while erecting. Gravel can be blown in later, while drilling.

It is not messy, like wet concrete.

It is cheap, involving low cost of labor and material.

It makes an effective fill.

Gravel packing is obligatory in connection with yielding support in swelling rock but otherwise it can be used anywhere except where the shed-water lagging is required for diverting water. Any type of tight lagging is suitable for being back filled with gravel. Any of the steel laggings, except the beam type, permit gravel to be blown through holes in the steel skin.

Grouting

Grouting to fill any spaces outside the concrete lining is usually done after the concrete is in place. But there are occasions where it is desirable to do grouting at low pressure soon after packing. Usually this is done only in connection with dry pack.

If grouting is to be done prior to concreting, liner plate lagging is required. It will hold grout at low pressures, especially if oats are added to the grout mix. If somewhat higher pressures are to be used, the liner plates can be fitted with gaskets. *Fig. 72* on page 141 shows a tunnel grouted in this manner. In the past, gaskets were commonly made of roofing felt. More recently, excellent gaskets have been developed of synthetic rubber.

CHAPTER 8

TUNNELS IN SOFT OR SQUEEZING GROUND

NORMAL AND EXCEPTIONAL TUNNELING CONDITIONS

Most of the footage of the existing rock tunnels has been constructed under the following conditions: The rock was excavated by blasting; the rock was strong enough to bridge the gap produced by firing a round at least until the heading was ventilated; the side pressure was zero or relatively low and the bottom of the tunnel was firm enough to support the footings of the ribs. These can be considered normal conditions in rock tunneling. The supports which are adapted to such conditions were described in the preceding chapters.

However, now and then, rock conditions are encountered which differ from the normal conditions in one or several respects. The most common deviations from normal are the following. The rock is so thoroughly crushed or decomposed that it does not require blasting. The tunnel may also intersect a buried valley filled with soft sediments. If such conditions are encountered, earth tunneling methods must be used. A brief description of these methods is contained in the next article. A more comprehensive treatment of the same subject will be presented in a companion volume "Earth Tunneling with Steel Support." Other abnormal rock conditions are known as squeezing and swelling conditions.

The term *squeezing rock* is applied to a rock or rock-derivative which squeezes into the tunnel wherever it is not held back by a tunnel support. A normal squeeze is nothing else but the rock deformation associated with the formation of the ground cylinder. It is exclusively due to the overburden pressure and to whatever residual horizontal pressures may act in the rock.

The squeezing rocks represent materials intermediate between rock and clay. Initially the rock may be hard enough to require blasting or it may be soft enough to be excavated with pneumatic spades. If they require blasting, rock tunneling methods can be used, but heavy side pressure combined with a tendency of the bottom to heave may require radical departure from the normal types of tunnel supports. If the squeeze is chiefly due to volume expansion, the squeezing rock constitutes a *swelling rock.*

EARTH TUNNELING IN ROCK TUNNELS

Factors determining earth tunneling technique

The technique of mining through earth depends chiefly on the bridge-action period of the earth above the roof and on the position of the tunnel with reference to the water table.

Tunneling through cohesive earth

Since no explosives are used excavation can immediately be followed by the construction of the support. Furthermore the distance between ribs seldom exceeds three feet. Hence if the earth has some cohesion, the roof exposed by mining ahead of the last rib can always stand up until the next support is erected, lagged and packed.

Liner Plate mining in raveling ground

The term *raveling ground* indicates a ground which can stand up for at least two or three minutes when an opening approximately 3 ft. wide by 16 in. forward is made. However, after this period, it may start to scale off. Slightly cohesive sand belongs in this category. In the past tunneling through this kind of ground was done by means of the forepoling method, which is slow, difficult and expensive. In recent times this method has been superseded by the more expeditious and economical liner plate method.

The installation of liner plates starts at the top center. The top breast board is removed and an opening just large enough to admit a liner plate, usually 16 in. x *37 11/16 in., is scraped out and the liner plate immediately shoved up against the ground. Its rear flange is bolted to the preceding course and a block is placed under its leading edge. The breast board is reinstalled 16 in. ahead. This process is repeated simultaneously for the plates down each side till each 16-in. course is complete. As the face is brought down, trench jacks are substituted for the blocks. A rib is installed under the course of plates and wedged, whereupon the jacks can be removed and the cycle started over again.

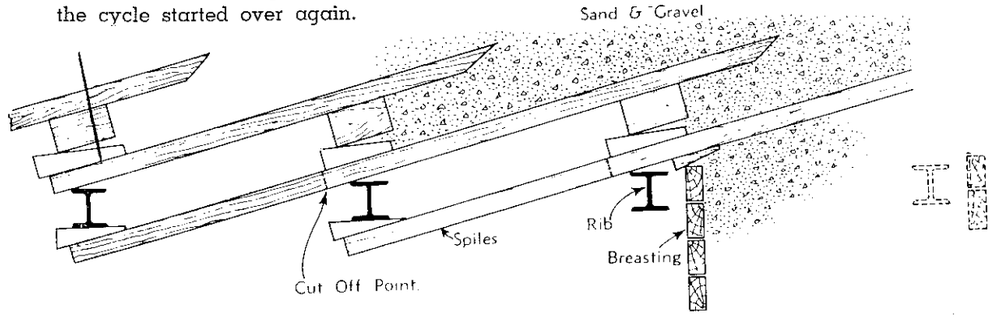


Fig. 104—Diagram of forepoling method of supporting running ground

The spiles are driven ahead before mining out. The breast boards are removed, one by one, the space ahead excavated and the breasting installed in its new forward position. Thus only a small portion of the face is open at any one time. The tails of the spiles are cut off prior to concreting.

Tunneling through running ground by forepoling method

Material with no cohesion, such as clean sand or gravel, is commonly referred to as *running ground* regardless of whether it is located below or above the water table. In running ground, excavation requires use of the ancient forepoling method or one of its modern equivalents. The working face requires support by breasting. *Fig. 104* illustrates the use of a modern forepoling method in combination with steel rib support.

The boards which are driven ahead to support the ground ahead of the last rib are known as *spiles*. They act as cantilevers which carry the weight of the ground until their forward ends are supported by installing the next rib.

The spiles are installed as far down around the sides of the tunnel as necessary. Then the top breast boards are removed, the exposed ground is excavated and the breast boards are reinstalled ahead as shown by the dotted lines in *Fig. 104*. This process is continued until the excavation arrives at floor level, whereupon the next rib

*37 11/16 inches = 3.1416 feet.

Fig. 105—Forepoling on steel ribs in sand.

Cantilever spiling is driven ahead into the sand at the crown. Short spiling, somewhat longer than the rib spacing, is driven in the haunches and sides where the sand will stand better. Upon completion of a course of spiling, the sand is excavated under protection of the spiling and the new rib set installed near the forward end of the spiles. The sand shown in this photograph stood up for vertical excavation; hence, no breasting was required.

Metropolitan Water District of Southern California
LOS ANGELES-COLORADO RIVER AQUEDUCT, SIERRA MADRE TUNNEL
near Pasadena, California

Contractor: J. F. Shea, Inc.
Tunnel dimensions: driven bore, 12'-7" by 12'-7", horseshoe; finished diameter, 10'-0".
Support: continuous ribs, 5" I-beams, 10 lbs.

can be installed. At some time afterwards the tails of the spiles are cut off. A forepoling operation is shown in *Fig. 105*. Spiles are commonly shoved in with the mucking machine. A smooth shove is better than a vibrating force.

Tunneling through earth below the water table

The seepage towards tunnels located below the water table increases very considerably the difficulties and cost of construction. It shortens or eliminates the bridge-action period, reduces the bearing capacity of the tunnel bottom and it may even cause a heave of the bottom. It also creates the danger of loss of ground due to soil being washed into the tunnel. Hence if an earth tunnel is located at a depth of less than about 80 ft. excavation is done under compressed air.

Fig. 106—Forepoling with Steel Spiles.

Spiling driven ahead of the face serves the same purpose as sheet piling in an open excavation. Movement of the ground above and at the sides of the tunnel is thus forestalled. In such cases the face usually requires supporting also.

In this tunnel channels were driven ahead in soft wet crushed shale. Ribs were erected as the face was advanced under this protection. Bottom struts were used to prevent sides from coming in.

CENTRALES HIDROELECTRICAS DE CANOAS AND EL COLEGIO
Near Bogota, Colombia, S.A.

Contractor: Morrison-Kundsen Co., Inc. (Construction EmKay, S.A.).

Tunnel Dimensions: 15'-9 1/8" wide by 16'-5 1/8" high, straight sides.

Support: 8" WF @ 31# horseshoe, spacing variable. Curved bottom struts 8" WF @ 31# were used in some areas to prevent bottom heaving and side squeeze.

The length of earth tunnel sections in rock tunnels seldom exceeds a few hundred feet. Mining through such short sections does not justify the expenditure involved in the installation of airlocks and a compressor plant. Furthermore, earth tunnel sections in rock tunnels are usually located at a depth of more than 80 ft. which precludes the use of the compressed air. Hence the engineer is usually compelled to mine through such sections without the aid of compressed air.

Free draining ground such as clean sand or gravel cannot be mined at all unless it is previously drained or grouted. Earth with a low permeability such as fine silty sand cannot be drained. Since the quantity of water which seeps into the tunnel out of such material is relatively small, the water does not interfere with the tunneling operations. However, the process of mining is extremely slow and expensive, because no more than very small areas can be exposed at a time.

In order to forestall runs the lagging elements of the support system must be driven into the ground before the ground is excavated as shown in *Fig. 106* where steel channels were used for spiles. These elements serve the same function as the sheet piles in open excavations in soft ground. This process of supporting the ground ahead of excavation is the forepoling method or an equivalent. Any but circular ribs seem to be out of place in such material because heavy side pressure is usually combined with soft bottom and heave.

EARTH TUNNELING CONDITIONS IN MONO CRATERS TUNNEL

General data

One of the most instructive examples of the occurrence of earth tunneling conditions in rock tunnels is the Mono Craters Tunnel near Leevining in California. The tunnel has a circular internal cross-section with a diameter of 9 ft. 7½ in. The total length of this tunnel is 59813 ft. Typical earth tunneling conditions were encountered in 9354 ft. out of this total. In these tunnel sections the method of forepoling on steel ribs such as shown in *Fig. 104* was used. Over a length of 7480 ft. out of the 9354 ft. breasting was required. In the balance of the tunnel rock tunneling conditions were encountered, but even within the rock, 30968 ft. of tunnel required support.

Geological and rock conditions

Fig. 107 shows a geological profile of the tunnel. The geological processes which led to the unfavorable rock conditions were described as follows:

"The eastern slope of the Sierra in this region is characterized by the presence of great glacial moraines extending from the base of the mountains far into the valley below.

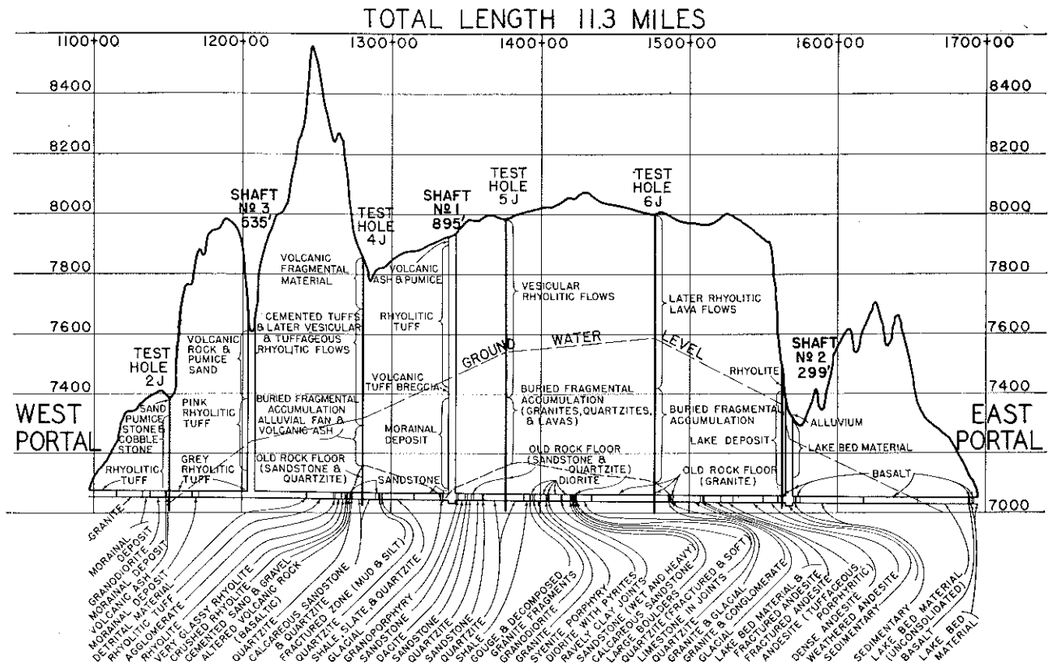


Fig. 107—Geological profile of Mono Craters Tunnel

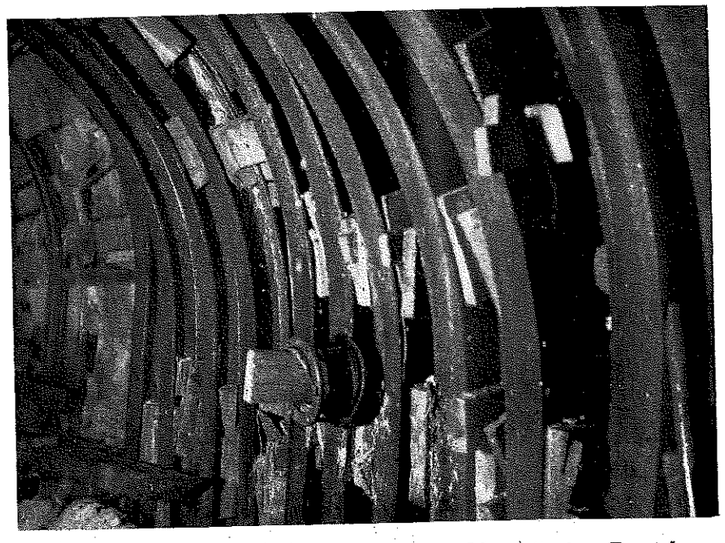


Fig. 108—Support in squeezing sections of Mono Craters Tunnel.

Steel ribs spaced 9 to 12 in. in squeezing section.

City of Los Angeles, Dept. of Water & Power
MONO BASIN PROJECT, MONO CRATERS TUNNEL
Leevining, California

Contractor: Force Account.
Tunnel dimensions: driven bore, 11′-9″ wide by 12′-3½″ high, horseshoe; finished diameter, 9′-7½″.
Support: continuous ribs and struts, 6″ H-beams, 20 lbs. spaced at variable centers, shown here at 9″ to 12″ spacing.

"To the south the boundary of the basin is partially formed by a series of eruptive volcanic cones, known as the Mono Craters, from which the tunnel derives its name. Some twenty in number, these volcanic peaks, attaining an elevation of 9,000 ft. or more, extend in a north and south direction for about 10 miles. Most of them are of very recent origin, being formed subsequently to the glacial flows that cover the ancient floor of the valley. For the most part, they are composed of acidic lava, ranging from fairly compact obsidian, a black volcanic glass, to light grey pumice. For a distance of approximately 10,000 ft., the tunnel penetrates these volcanic cones.

"The mesa separating the basin and the Owens River along the line of the tunnel for about 8 miles has an elevation of approximately 8,000 feet.

"The mesa occupies part of what was once a deep valley lying between the Sierra on the west and the White and Inyo Mountains to the east, each range rising to an elevation of 13,000 ft. or more. Portions of the floor of this ancient valley were covered with lakes which, at a later period, were filled with products of erosion from the steep slopes of the surrounding mountains.

"During the quaternary period great glaciers, of which remnants are still to be found, existed in the High Sierra forming the west and southwest boundary of the Basin. In time, these ice and snow fields with included debris built great moraines at the base of the mountains and far out into the valley floor. At a later period, the region was visited by great volcanic

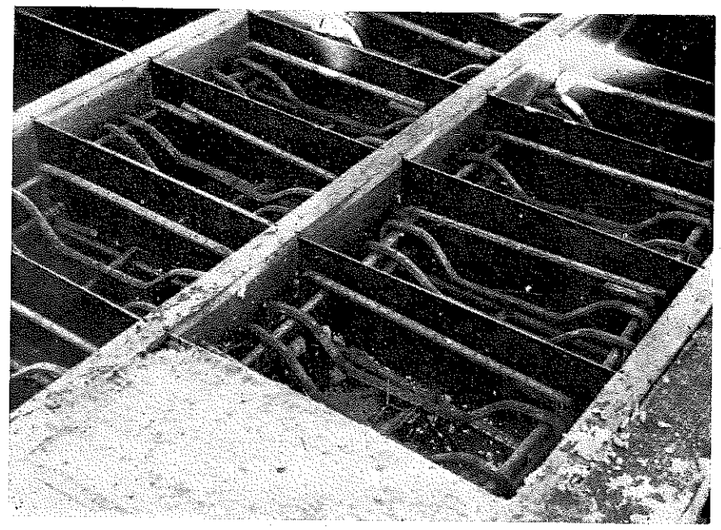


Fig. 109—Precast concrete foot blocks.

Tunnel supports are no better than the footings under the ribs. In earth sections and fault zones frequently encountered in rock tunnels, the customary wood blocking may not satisfactorily spread the load over the soft ground. Precast concrete foot blocks have proven quite satisfactory for this purpose.

This photograph shows forms and reinforcing for concrete foot blocks as used in the Mono Craters Tunnel. Used in two sizes, 12 x 18 x 24 in. and 9 x 12 x 18 in. these blocks satisfactorily distributed the heavy rib loads to the earth bottom.

activity resulting in the formation of high volcanic peaks, of which the Mono Craters are a part and the covering of the surrounding terrain for miles with lava and volcanic ash.

"The Mono Craters Tunnel from portal to portal is driven through such formations."*

Tunneling conditions

It is not surprising that nearly every difficulty inherent in rock tunneling was encountered in driving this tunnel. Large volumes of water under high head were encountered both in the rock and earth stretches. The water was charged with gas presenting an extra difficult problem of ventilation. In many places holes were drilled deep into the rock to intercept the water which was bled to the pipe drains by closed piping thus preventing it from liberating its dissolved gas.

Squeezing ground was encountered at several places requiring closely spaced ribs and invert struts. See *Fig. 108*. In the earth tunnel stretches the loads were so great that reinforced concrete foot blocks, 12 x 18 x 24 in. and 9 x 12 x 18 in. as shown in *Fig. 109* were required under the ribs. These proved so satisfactory that they were used quite generally in the rock sections. Yet with all the difficult going some favorable stretches were encountered. In one section through cemented volcanic ash, one heading was advanced 90 ft. in 16 hours.

In this tunnel as shown in *Fig. 110*, spiles were 4 x 6 or 6 x 6 in. timbers and in some stretches were 6-in. steel channels and were used even below spring line.

* A paper presented on October 26, 1939, at the California Section Meeting of the American Water Works Association in San Francisco, by H. L. Jacques, Engineer of Major Construction, Bureau of Water Works and Supply, Department of Water and Power, Los Angeles.

Fig. 110—Forepoling in Mono Craters Tunnel.

Of the 59,812 ft. of this tunnel, 8354 ft. were driven by forepoling through morainal deposit, sand and gravel, crushed rock, mud and silt, fault gouge, and clay. Average cover was about 1000 ft. which contained ground water with a head ranging up to 600 ft.

Spiling was 3 x 6, and 6 x 6 in., used the 6 in. way as shown in the photograph. Spiles were generally driven by using the long timber shown in the foreground as a battering ram, and bumping it with the mucking machine.

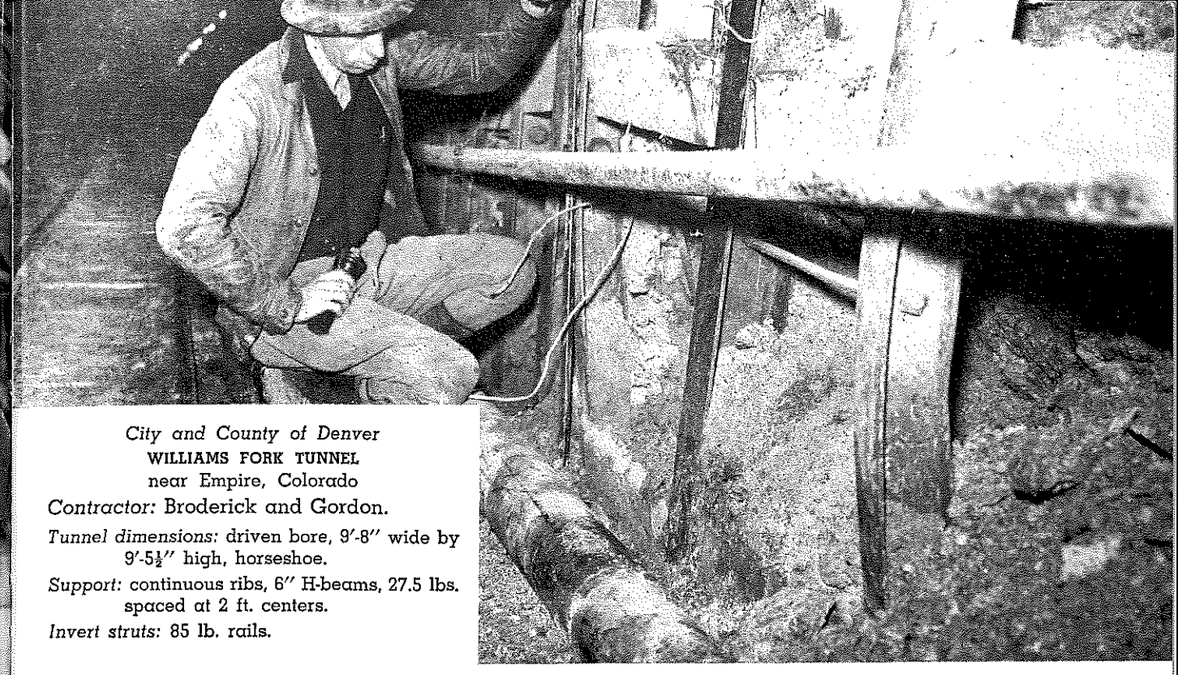
This photograph shows a set of spiles partially driven, those in the crown advanced further than the ones in the haunches. Later, the tails of the spiles were cut off at the rib.

City of Los Angeles, Dept. of Water & Power
MONO BASIN PROJECT, MONO CRATERS TUNNEL
Leevining, California

Contractor: Force Account.

Tunnel dimensions: driven bore, 11′-9″ wide by 12′-3½″ high, horseshoe; finished diameter, 9′-7½″.

Support: continuous ribs, 6″ H-beams, 20 lbs.; normal spacing 5′-0″ but frequently used at much closer spacing, sometimes down to 9″.



City and County of Denver
WILLIAMS FORK TUNNEL
near Empire, Colorado

Contractor: Broderick and Gordon.

Tunnel dimensions: driven bore, 9′-8″ wide by 9′-5½″ high, horseshoe.

Support: continuous ribs, 6″ H-beams, 27.5 lbs. spaced at 2 ft. centers.

Invert struts: 85 lb. rails.

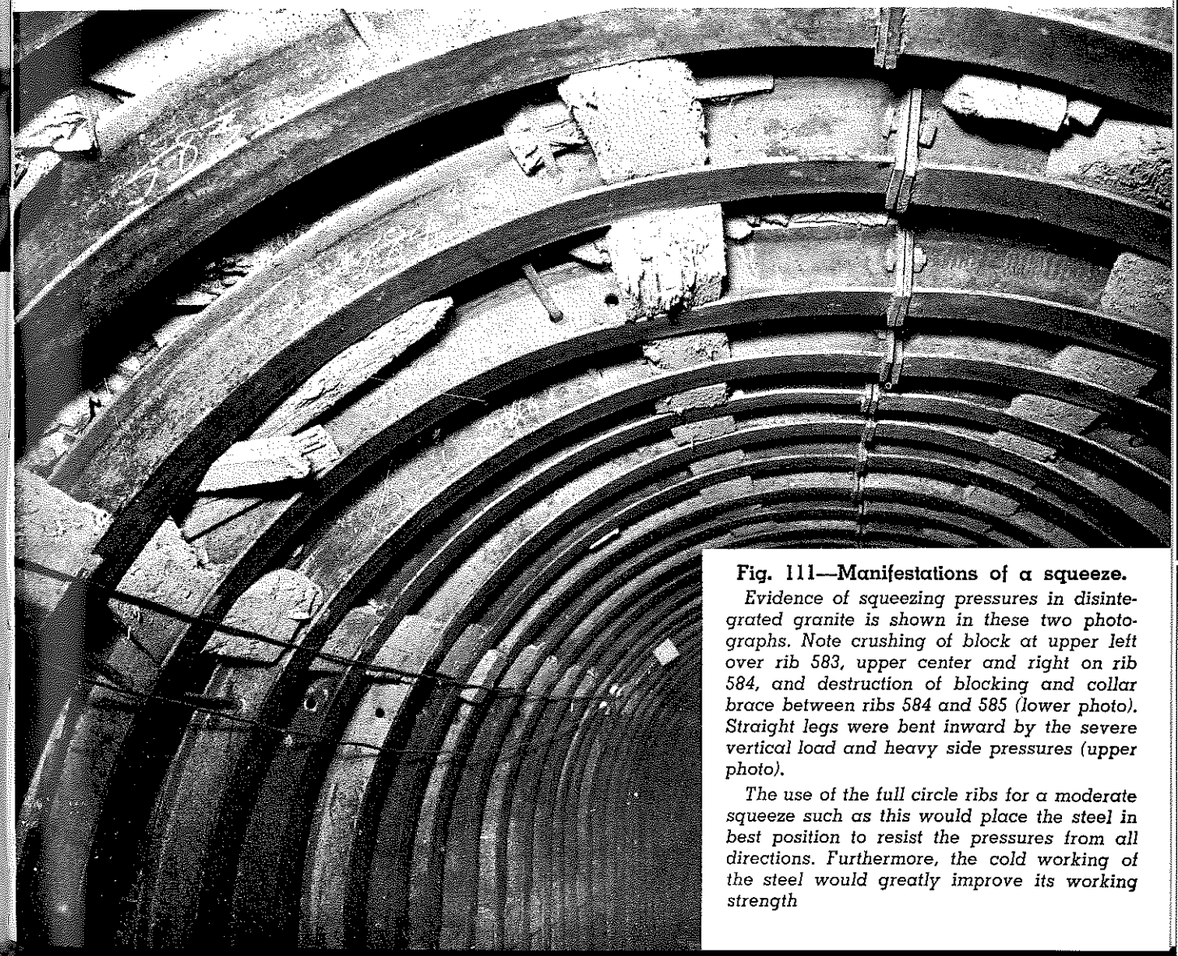


Fig. 111—Manifestations of a squeeze.

Evidence of squeezing pressures in disintegrated granite is shown in these two photographs. Note crushing of block at upper left over rib 583, upper center and right on rib 584, and destruction of blocking and collar brace between ribs 584 and 585 (lower photo). Straight legs were bent inward by the severe vertical load and heavy side pressures (upper photo).

The use of the full circle ribs for a moderate squeeze such as this would place the steel in best position to resist the pressures from all directions. Furthermore, the cold working of the steel would greatly improve its working strength

MINING THROUGH SQUEEZING GROUND

Squeezing ground exerts pressure onto the tunnel support from all sides as clearly shown in *Fig. 111*. Therefore if such ground is encountered, it is highly desirable to convert the external cross-section of the tunnel to a full circle even if it means enlarging considerably to maintain the internal clearances. The steel ribs then act as rings and are subject only to thrust, and not bending, thereby deriving the greatest resistance per pound of steel used. Furthermore all the steel in the ribs is cold worked thereby increasing the stresses that may safely be applied. (See *chapter 9*).

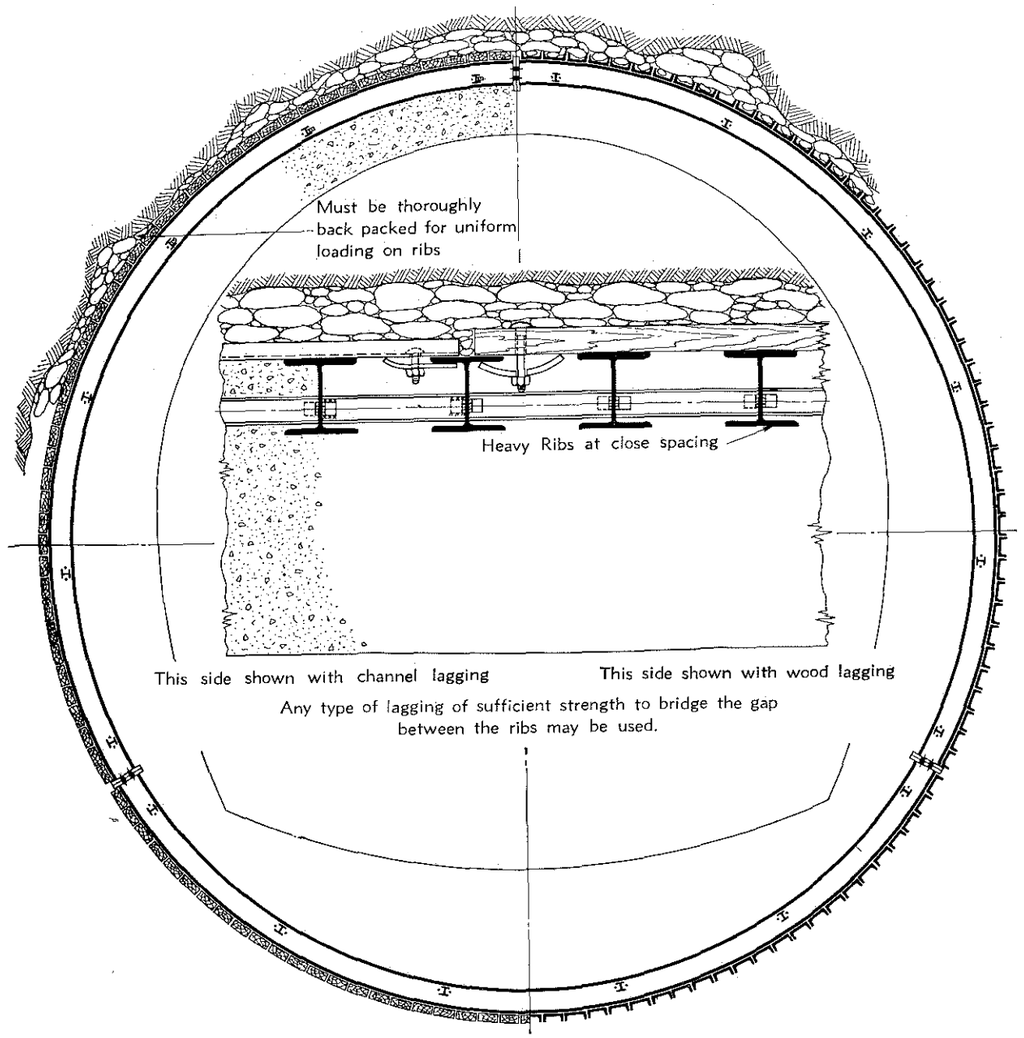

Must be thoroughly
back packed for uniform
loading on ribs
Heavy Ribs at close spacing
This side shown with channel lagging
This side shown with wood lagging
Any type of lagging of sufficient strength to bridge the gap
between the ribs may be used.

Fig. 112—Full Circle Ribs closely spaced and heavily lagged for heavy loads associated with squeezing conditions

The full circle ribs should be closely spaced and a tight lagging of sufficient strength to transfer the estimated load onto the ribs should be placed on the outside flanges as shown in *Fig. 112*. The space between the lagging and the ground *must be thoroughly filled* initially to establish complete contact between the ribs and the ground, otherwise the circular ribs will not function at their rated capacity.

Dry pack or dry concrete may be used behind tight wood, channel, beam, beam and plate, liner plate, or purlin type lagging of proper strength.

Dry pack *grouted* or wet concrete placed by machine require liner plate lagging. (Note: This packing constitutes a shell of concrete around the supporting structure that adds materially to the strength of the whole structure).

Gravel, blown in, most effectively through liner plate or a purlin type lagging, is a cheap and effective method of filling and can be placed while other operations are being performed.

The steel ribs should be considered to be the permanent lining. The concrete, placed much later, is the preserving medium although it will receive part of any increases of load after it has cured, thereby adding to the ultimate factor of safety of the completed structure.

MINING THROUGH SWELLING GROUND

Methods of mining through swelling ground

If the rock has a moderate swelling capacity the same method of supporting the rock should be used as in squeezing ground. It involves the installation of rigid, circular ribs immediately followed by thorough back packing. On the other hand, if the rock has a high swelling capacity, it may be advantageous to give it an opportunity to expand into the tunnel during the entire service period of the tunnel support.

In the past this method was practiced by permitting the rock to crush one or more timber sets in succession and then to construct the permanent lining in such a manner that a clearance of about 6 inches was left between the extrados of the lining and the rock. Since the inward movement of the rock during the preceding failure of at least one set of timbering in a large tunnel was hardly less than 6 inches, the total yield of the rock walls toward such a tunnel was at least one foot in every radial direction.

The replacement of one or more sets of timbering is very bothersome and expensive and interferes with other operations. In the United States it is customary to meet a threatened failure of the tunnel support by constructing the permanent lining in the swelling stretch without further delay. This procedure has the disadvantage that it reduces very considerably the cross-section of the tunnel which may create a bottleneck. It also involves the risk of underestimating the ultimate pressure on the permanent lining. If the permanent lining fails the consequences may be very serious. In order to prevent a failure of the temporary support, the support may be made up of rigid circular ribs interconnected by a yielding lagging which permits the rock to squeeze into the tunnel through the spaces between the ribs. The yielding lagging can also be supported by yielding ribs consisting of steel segments separated from each other by compressible blocks.

Yielding lagging

The fundamental principle of this method is to install heavy full circle ribs at wide spacing, say 4 feet, and let the ground squeeze between the ribs till it relieves itself, which means that ground cylinder action has been established. This method can be used

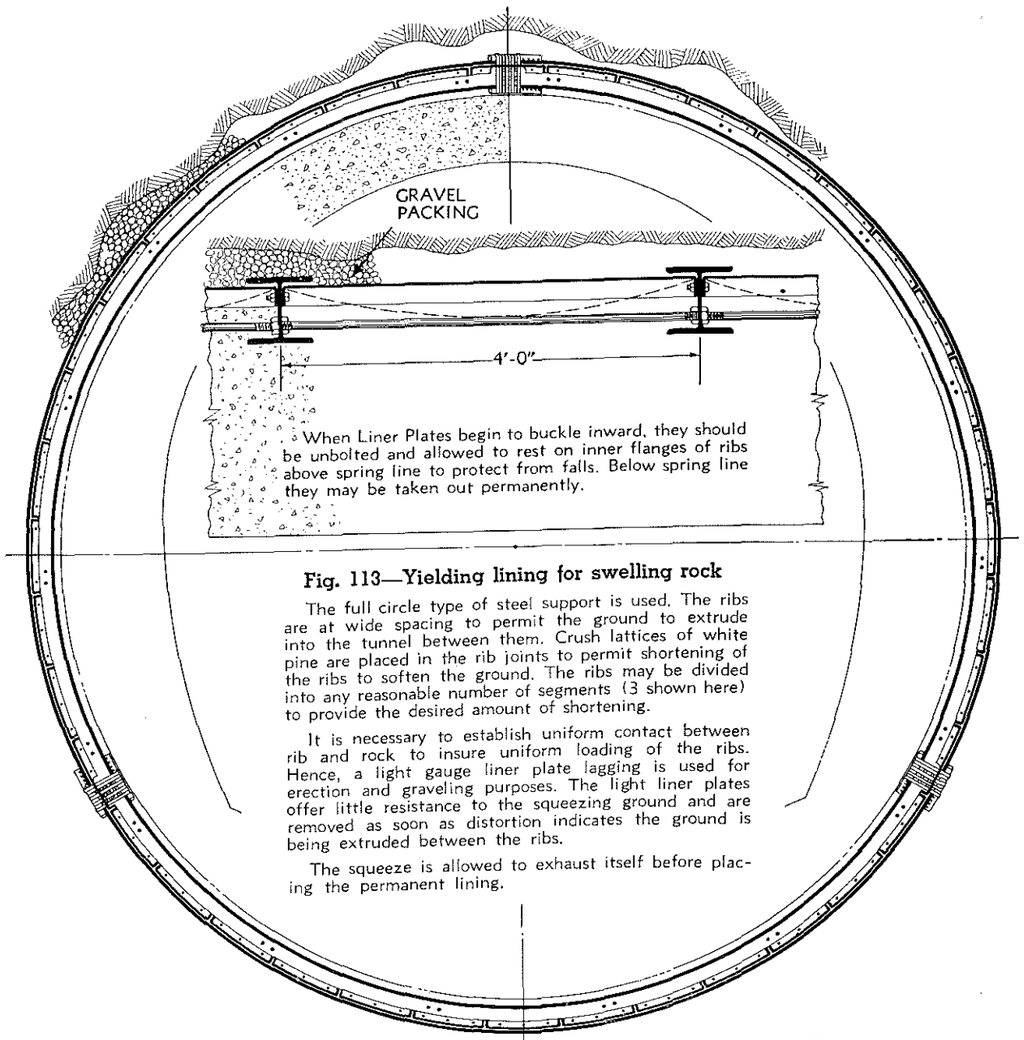
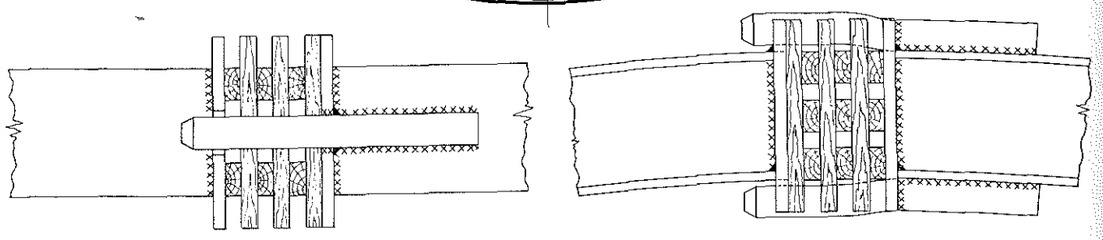

GRAVEL
PACKING
4'-0"
When Liner Plates begin to buckle inward, they should be unbolted and allowed to rest on inner flanges of ribs above spring line to protect from falls. Below spring line they may be taken out permanently.

Fig. 113—Yielding lining for swelling rock

The full circle type of steel support is used. The ribs are at wide spacing to permit the ground to extrude into the tunnel between them. Crush lattices of white pine are placed in the rib joints to permit shortening of the ribs to soften the ground. The ribs may be divided into any reasonable number of segments (3 shown here) to provide the desired amount of shortening.

It is necessary to establish uniform contact between rib and rock to insure uniform loading of the ribs. Hence, a light gauge liner plate lagging is used for erection and graveling purposes. The light liner plates offer little resistance to the squeezing ground and are removed as soon as distortion indicates the ground is being extruded between the ribs.

The squeeze is allowed to exhaust itself before placing the permanent lining.



DETAIL OF CRUSH LATTICE

in those cases where the strength of the rock is low enough to squeeze through without overstressing the ribs. The ribs should be of quite heavy cross-section as the pressures are great and the ribs are widely spaced. The clearance around the tunnel should be somewhat greater than called for by the design to allow for some distortion and displacement of the rib without encroachment on the design line.

An essential requirement is to establish without delay complete and intimate contact between the ribs and the rock. This is best accomplished by installing liner plates (of light gauge) between the ribs and blowing gravel behind. The ribs should not be blocked at any point. The blocking necessary to install the rib and liner plate lining should be placed against the liner plates so that no concentrated loads will act on the ribs. This precaution serves to make sure that ribs act in compression only.

When the squeeze develops, the gravel is trapped between the rock and the outer flange of the ribs, thereby loading the ribs uniformly. The liner plates will start to buckle inward as indicated in *Fig. 113*, at which time they can be unbolted from the webs of the ribs. The plates below the spring line can be removed permanently and the crown plates can be left resting on the inner rib flange to protect the operations from falling rock as shown in *Fig 114*. The progress of the squeeze should be closely watched. When it becomes evident that the space behind the liner plates (now resting on the inner flange) is completely filled up, the plates should be removed, the muck taken out to the back of the ribs or further, and the liner plates laid back in. This operation should be repeated till the time for placing the permanent concrete lining has arrived.

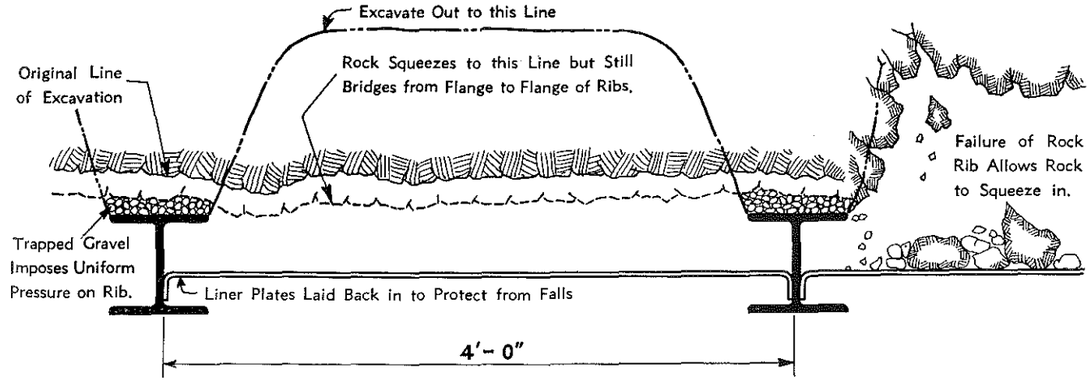

Excavate Out to this Line
Original Line of Excavation
Rock Squeezes to this Line but Still Bridges from Flange to Flange of Ribs.
Failure of Rock Rib Allows Rock to Squeeze in.
Trapped Gravel Imposes Uniform Pressure on Rib.
Liner Plates Laid Back in to Protect from Falls
4'- 0"

Fig. 114—Over-mining in stiff swelling ground to induce softening

In stiff swelling ground it is necessary to allow the ground to squeeze some undetermined amount to soften it. If the ground is not sufficiently soft to extrude between the ribs when the shrinkage provided in the crush lattices (Fig. 113) is used up, slots are excavated beyond the ribs, as shown here. This is repeated until extrusion between the ribs is established. The squeeze is then allowed to run its course before concreting.

Yielding ribs and lagging

If the squeezing rock is so firm that it would arch from rib to rib and thus overstress the ribs, it would be advisable to use a yielding rib to enable the rock to make its initial movement. As the rock advances into the tunnel, its strength decreases until it becomes soft enough to squeeze between the ribs.

The ribs are spaced about 4 ft. Each rib is made out of several segments. Between the butt plates of each two adjacent segments a *Crush Lattice* of white pine strips

is inserted as shown in *Fig. 113*. The space between the ribs is bridged by light gauge liner plates. The lining is backfilled with pea gravel as described under the subheading "yielding lagging."

The purpose of the crush lattice is to allow the diameter of the ribs to decrease and thereby permit the rock to move toward the tunnel. After the diameter of the ribs has decreased as much as the "crush lattice" permits a few liner plates are removed to inspect the condition of the rock. If the rock has become soft enough to squeeze through the spaces between the ribs the subsequent procedure is identical with the one described under the subheading "yielding lagging." On the other hand, if the rock is still too stiff to squeeze through the gaps between ribs, all the other liner plates are removed and the rock between ribs is excavated as shown in *Fig. 114*, leaving rock ribs between grooves. Every steel rib is in contact with a rock rib. Then the liner plates are laid back in on the inner flanges in the crown to prevent rock fragments from dropping.

As the rock continues to swell the rock ribs adjoining the outer flanges of the steel ribs are crushed and the debris fills the spaces behind the liner plates. After the spaces are filled and the liner plates have bulged as far as their dimensions permit, the liner plates are removed and new grooves are excavated. Thus it is possible to permit even in a very stiff swelling rock a large amount of squeeze into the tunnel while the ribs remain intact.

As the crush lattices are compressed the diameter of the ribs decreases. Subsequent unequal failure of the rock ribs may even slightly deform the steel ribs or displace the whole rib set. Hence the initial diameter of the ribs should be such, that at the end of the service period, the ribs are still located at or beyond the position provided for by the construction drawings. The dimensions of the space which has to be excavated is determined not by the final but by the initial diameter of the ribs.

Advantage of yielding tunnel support

In the United States, the cost of labor in tunnel construction usually exceeds very considerably the cost of material. Therefore, it is to be expected that the prevention of a failure of the ribs at the price of installing heavy profiles is cheaper and more expedient than to clean up the tunnel after complete breakdowns of timbered support. If the ribs are strong enough, it will not be necessary to place the permanent lining till the rest of the tunnel can also be concreted. This avoids the necessity for the premature setting up of concrete plant and placing of a relatively small amount of concrete at a high cost. It also avoids a constriction of the tunnel in the squeezing stretch.

Rock pressure measurement

No matter which method is used some of the ribs should be provided with sturdy pressure measuring devices which permit periodic determinations of the pressure on the ribs. The information thus obtained would reduce the amount of guesswork which enters the design of the permanent lining.

Benefits derived from pilot drifts

In *Chapter 5* it was shown that the pressures exerted by squeezing rock depends to a large extent on the condition and the nature of the rock. Yet as a rule, the geologist cannot even predict with certainty in advance of construction whether any squeezing rock will be encountered at all. A prediction of the intensity of the pressure is utterly beyond the scope of rational forecasts. Hence if a tunnel with a large cross-section is

to be built through folded or faulted rock involving the possibility of encountering squeezing rock the design of both the tunnel support and the permanent lining can only be based on a guess and the consequences of a wrong guess may delay construction very considerably.

Pilot drifts almost eliminate these hazards. The results of the observations in the pilot drift inform the engineer as to whether the bad rock is of the squeezing or the swelling type; they furnish the data required for deciding whether or not it will be advantageous to use full circle ribs. Last but not least, the pilot drift gives to the engineer an opportunity to measure pressures and the rate of pressure increase. The results of these measurements eliminate much of the guess work from the design of the supports in the main excavation. Since the information obtainable is of such vital importance the advantages of pilot drifts on tunnel jobs in geologically disturbed regions cannot possibly be overemphasized.

CONVERSION TO CIRCULAR SECTIONS

General considerations

In running ground and in squeezing or swelling rock, tunnel supports with full circle ribs are so perfectly adapted to meet the pressure conditions that all the other types of support can only be considered unsatisfactory makeshifts. Although the conversion from the normal type of rock tunnel support to supports with full circle ribs requires some extra labor and material, these extras are likely to be unimportant compared to the difficulties and delays associated with adhering to the normal types of tunnel supports in difficult stretches.

Hence, even if short sections of a tunnel are located within running or squeezing ground, recourse to tunnel supports with full circle ribs for the bad sections should always be seriously considered. Such a study will most likely show that the conversion is by no means as difficult as it may appear at superficial examination.

In addition to the fact that full circle ribs are better suited than any others to meet the pressure conditions they also serve as a reinforcement for the permanent lining. Since the outer face of the permanent lining constructed in a tunnel stretch with full circle ribs is cylindrical the bending moments in the lining are very small and the strength of the construction material is utilized to full advantage. In bad swelling ground the additional strength furnished by the ribs encased in the concrete may even be required, because the pressure on the lining continues to increase for a long time after the lining is constructed.

Local changes of the tunnel dimensions

The benefits of the full circle rib support in ground that develops side pressure derive from conversion of the extrados to a circle. This may or may not be accompanied by a conversion of the intrados to a circle, depending on economic and other factors.

In making the conversion of the extrados, the inner diameter of the ribs should be so chosen that clearances required by the excavating equipment are maintained. Generally this involves an increase of the yardage of excavation per foot of tunnel. If the intrados is to remain unchanged, a corresponding increase in concrete yardage is also involved. If the intrados is to be converted to a circular cross section, a new set of forms is required but the yardage of concrete is often reduced. A short stretch would not justify the cost of a set of forms whereas in a longer stretch the savings of concrete would overbalance the form cost.

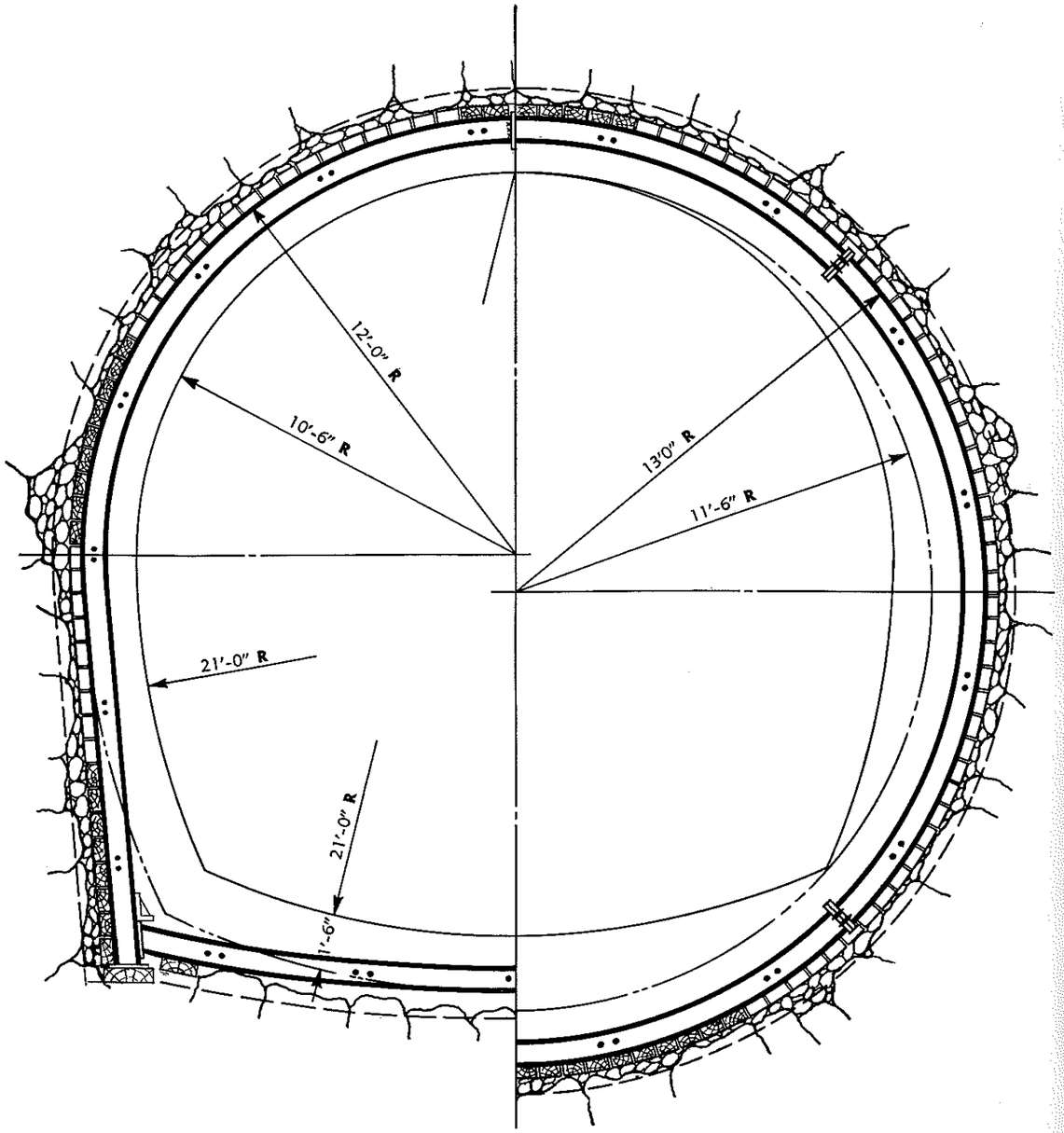


Fig. 115—Conversion of a horseshoe shaped flow tunnel to a circular shape in squeezing ground

The following example is a comparison of a horseshoe and a circular section through squeezing ground period.

Conversion of a flow tunnel

The left side of Fig. 115 shows the standard, and the right side the modified circular cross-section of a flow tunnel. A comparison of quantities is made in the following table. It is assumed that the tunnel is driven through squeezing ground requiring tight lagging.

	Horseshoe	*Circular*
External dimensions - - - - - -	24′ x 24′	26′ dia.
Excavation to overbreak line in cu. yds. per foot - - - - - -	20.1	21.6
Concrete to back of ribs in cu. yd. per foot		
Inside unchanged - - - - -	5.1	6.2
Inside circular - - - - -		4.3
Weight of steel per foot of tunnel - - -	662 lbs.	665 lbs.
Carrying capacity of 8″ x 8″ beam, 31 lbs. on 4 ft. centers, rock, 170 lbs. per cu. ft.		
Vertical load only, no side pressure - - - - - - -	23.8 ft.* of rock	23.5 ft. of rock
Horizontal unit pressure equal to 1/3 of the vertical unit pressure -	13.9 ft. * of rock	23.5 ft. of rock

* Computations are set forth as *Example No. 4* in *Chapter 11.*

A small horizontal load makes a large reduction in the capacity of a set with a straight leg. As shown by the table above if the horizontal pressure is 1/3 of the vertical (as applied to the particular set of dimensions in *Fig. 115)* the capacity of the rib decreases by 42% whereas the capacity of a full circle rib remains unchanged. In fact due to cold working the fibre stress allowed for the full circle rib could be safely increased by 25% and thereby gain that much in capacity. (See *Chapter 9).*

When side pressure is encountered it is readily seen from the above comparison that:

1. A pound of steel support in the form of a full circle rib will carry almost twice the load it will carry in the form of a straight legged rib.

2. That if the length of the squeezing stretch is long enough to justify the forms, a saving of concrete results. Note: The regular forms may often be temporarily padded out to the circular shape at a nominal cost.

3. The above advantages and others can be secured at a very small increase in the excavation cost.

Conversion of single track railroad tunnel

At first glance it would appear that a single track railroad tunnel would not show any economies when converted to a full circular section. Yet when squeezing conditions, which produce heavy side pressures, are encountered, conversion to the circular cross-section is especially desirable. This is clearly shown in the tabulation of quant-

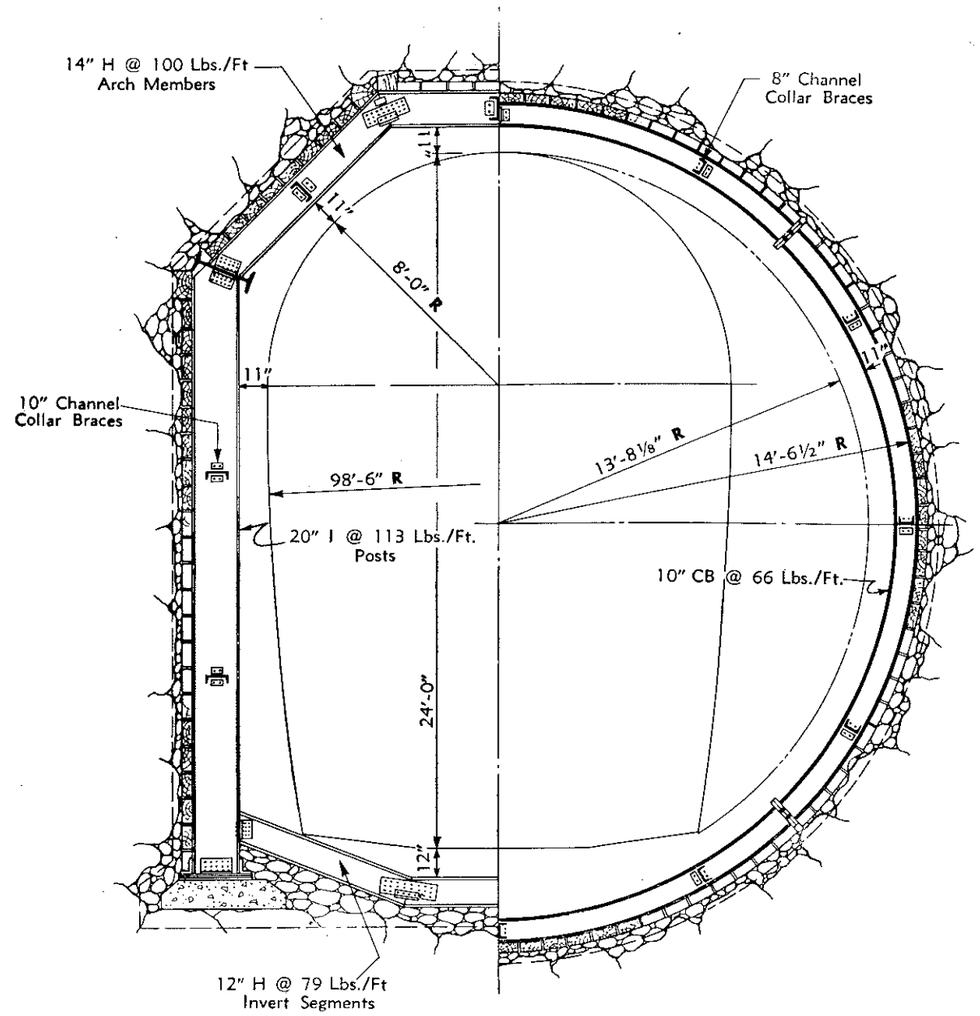


Fig. 116—A study of the advantages of conversion of the single track Moffat Tunnel to a circular cross-section

The "heavy" 8-piece steel sets that finally held the swelling squeeze after failure of timbers and lighter steel sets are shown at the left. A full circle rib of equal strength appears at the right.

The table of comparative quantities on the next page indicates a saving in favor of conversion to the circular shape. But if the technique of the yielding lining had been available at the time this tunnel was built, far greater savings could have been made. Replacement of timber and steel sets (as many as 5 times in some stretches) which cost enormous sums, could largely have been avoided.

The adjacent 8 by 8 ft. water tunnel, after repeated failure of skin tight timbering was enlarged to 13 ft. diameter and lined with circular steel support with complete success.

ities immediately following which is compiled from a study of conversion of the Moffat Tunnel in Colorado to a full circle, making the comparison on the basis of computed strength of the steel support used.

The left side of *Fig. 116* shows the "heavy" steel sets used in the Moffat Tunnel. The right side shows a full circle set of equivalent strength. The stresses in the various members of the eight piece steel set are approximately equal when the side pressure is about 20% of the vertical pressure. In making the comparison, the fibre stress in the steel was considered the same in both types of support although in the full circle ribs the stress could be higher since the elastic limit of the steel is raised by cold working.

	Cross-section as constructed	*Circular Cross-section*
External dimensions	21'-2"x28'-1"	29'-1" dia.
Excavation to overbreak line in cu. yds. per ft.	22.2	26.3
Concrete to back of ribs in cu. yds. per ft.	8.1	6.0 *
Weight of steel in pounds per ft.	3666	2166
Carrying capacity of sets assuming side pressure is 20% of the vertical pressure. Fibre stress 24000 p.s.i., rock 170 lbs. per cu. ft., 3 ft. spacing	59 ft. of rock	59 ft. of rock

* Finished bore circular, 27'-4¼" dia.

An analysis of *Fig. 116* and the above table, reveals that for an increase of only 19% of the excavation, the following favorable results accrue.

a. A saving of 25% of the concrete.

b. A saving of 41% of the steel support.

c. A much lower cost of erection of the steel support due to the simple butt joints. A complete 4-piece rib as indicated requires only 16 bolts in the ring, whereas the 8-piece set used requires 216 bolts in the spliced joints.

d. The factor of safety is increased because the circular ribs have been cold worked.

The evaluation of the above factors shows that the conversion to the circular cross-section results in considerable economy. But these factors are only the quantitative factors, they do not take into account the trouble and expense that can undoubtedly be saved if the full circle ribs are used to support the original excavation.

Conditions of extremely heavy pressures such as were encountered in the Moffat Tunnel for about 2½ miles through fault zones and associated decomposed squeezing granite indicated the need of full circle ribs. These conditions were as described in *Chapter 4* on page 80.

At the time this tunnel was driven full circle ribs had not yet been developed for rock tunnels, although full circle metallic linings had been widely used in earth tunnels. Therefore timber sets, or steel sets of similar form, were the regularly used types of support for rock tunnels.

Following normal practice of the times, timber sets of 12x12 in. size were originally installed in the tunnel about 3 ft. on centers. Some "jump" sets were placed between these when the timber showed signs of being highly stressed. This proved inadequate as the constantly increasing pressures crushed and broke the 12x12 in. timbers even where they were placed skin tight.

After some experimentation, these sets were replaced with 12x18 in. timber much of it being skin tight, making a solid timber lining 18-in. thick. This also proved insufficient and was replaced with structural steel sets of fairly similar pattern, spaced 3 ft. on centers. These first, or "light," steel sets consisted of a 3-segment roof of 20-in. 65.4-lb. I-beams supported on 24-in. 80-lb. posts with an I-beam wall plate lying flatwise, and 3 18-in. 57.4-lb. invert segments between the posts at the bottom.

These sets also proved inadequate, even though "jump" sets were placed between, making the spacing 18 in. on centers. A "heavy" steel set was then designed which held the ground. It is shown at the left of center line in *Fig. 116*. These sets are supported by precast reinforced concrete foot blocks to minimize settlement and 10-in. channel collar braces were concreted in as soon as bolted to prevent twisting. Even with these heavy steel sets some posts had to be replaced but in general they proved strong enough to hold the ground.

Until the "heavy" sets were installed, the ground had continued to invade the tunnel bore, necessitating replacement of supports. This inward movement permitted the surrounding ground to partially relax, establishing a "ground cylinder" around the tunnel bore as described in *Chapter 4* thereby limiting pressure on the supports. Hence the "heavy " steel sets which were capable of resisting side pressure of only 20% of the vertical intensity were ample. Initial side pressure must have been much higher.

Full circle steel ribs can hold side pressure equal to the vertical pressure. Had this type rib and the technique of using them, as described in this chapter been available at the time this tunnel was driven, there is no doubt that other important savings could have been made in addition to those set forth in the above tabulation.

The hypothetical horseshoe tunnel, *Fig. 115*, and the railroad tunnel, *Fig. 116*, are probably the extremes. Most other tunnel cross-sections would lie between the two. Hence as a general rule, when side pressure or squeezing conditions are encountered, the circular cross-section with full circle ribs results in the lowest cost structure.

SECTION III

DIMENSIONING OF TUNNEL SUPPORTS

CHAPTER 9

BASIC CONCEPTS AND DATA

APPLICATION OF LOADS

Differences between a free arch and a tunnel arch

The major portion of the overburden pressure in a rock tunnel is carried by a ground arch which, under static conditions, develops in the rock located above the tunnel. The thickness of the ground arch varies according to the type of ground, size and depth of the tunnel, method of driving, and the amount of support which is provided in the excavated opening.

Obviously, an arch is the natural shape for a tunnel roof, and the problem of the designer is to supply a support system with enough strength to carry the weight of the keystone of the natural arch.

The roof structure is wedged against a mass of rock or earth which cannot be thrust outward from the structure. Therefore, the problem of designing an arch rib for a tunnel is quite different from that of designing an arch out in the open.

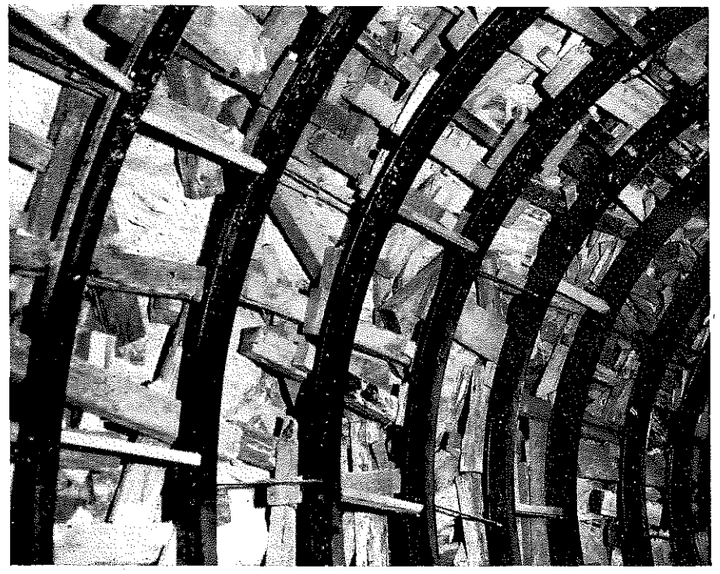


Fig. 117—Typical blocking of ribs.

Irregular spacing of blocks is inherent in tunnel construction. The curved steel rib makes it possible to block at the more favorable spots in the overbreak.

Metropolitan Water District of Southern California
LOS ANGELES-COLORADO RIVER AQUEDUCT, WEST EAGLE MOUNTAIN TUNNEL
near Mecca, California

Contractor: Broderick and Gordon

Tunnel dimensions: driven bore, 18′-2″ wide by 17′-10″ high, horseshoe; finished bore, 16′-0″ by 16′-0″ horseshoe.

Support: continuous ribs, 6″ I-beams, 12.5 lbs., at 3 ft. centers.

In designing an arched structure above ground, the engineer will usually adjust the shape of the arch to follow a thrust line established from known loads, acting at known points, in known directions. The arch will be supported at the two ends only, by the passive resistance of the abutments.

In a tunnel, the shape of the arch rib is determined not by the active loads but by the tunnel cross-section and may depart widely from a thrust line established from *assumed active loads*. Nevertheless the thrust line always can be made to follow the arch contour required. This is possible because the *passive resistance* of the surrounding rock resists any force tending to displace it. This passive force will prevent ribs from changing shape when properly blocked and wedged.

Active loads tend to move or distort the rib, and are caused by the weight of rock fragments that become loose and tend to drop out of the roof or side walls.

Passive resistance prevents rib distortion, and can be mobilized in any required magnitude by blocking between the rib and unyielding rock.

Blocked tunnel ribs

If a tunnel is provided with a tight, back-packed lagging, the rock acts uniformly on the entire tunnel rib, exerting either active or passive force. This, however, is not the ordinary condition. In most instances the rock forces are transferred to the ribs by a relatively small number of blocks which are inserted between the rock and the outside of the ribs. The places where these blocks bear on the rib are known as *blocking points*. Blocked ribs are shown in *Fig. 117*.

At any blocking point the rib is acted upon by an external force, which may be *active* (pressure of the rock against the rib) or *passive* (resistance of the rock to rib pressure). The block is capable of transmitting force to or from the rib only in a direction normal to the tangent at its point of contact with the rib. This is due to the fact that the block acts like a cushion with relatively small resistance to shear deformation. Hence any active loads, usually vertical, are resolved into two components, one of which is radial. Only this radial (or normal) component is considered. The resolution of forces is explained in detail further on.

LOAD ASSUMPTION

The designer will first assume an active load and apportion it to the various blocks. In the absence of any better assumption he usually will assume that the steel support system will have to carry a certain thickness of rock above the tunnel and that this rock will impose uniform vertical active loading on the ribs. The following discussion will show this to be a practical assumption.

He may question in his own mind the uniformity of the load distribution and with good cause, because it is a well known fact that rock falls usually leave an opening shaped like an inverted "V" or "U." It might seem that this would impose a concentrated load which would upset the assumption. However, as shown further on in the text, the active rock force at some one blocking point will determine the thrust in the rib. The thrust induced by this one force causes the rib to press against the rock at all other blocking points with a force greater than the active rock forces at those points. Hence, the rib is actually proportioned for a single concentrated load on one block. All other active loads can be neglected when this is found.

Inasmuch as it cannot be determined which of the blocks around the periphery of the rib will be the one to induce the maximum thrust condition, the loading is assumed

to be uniform. The thickness of the rock assumed to be acting uniformly will be ample to cover the maximum number of feet of rock anticipated to be active at any one blocking point. Thus in stratified rock as shown in *Fig. 19 c*, page 54, the actual weight of the rock to be carried by the support in a tunnel with width **B** is the weight of the rock in the inverted "V" whose depth is 0.5**B**. The stresses in the rib, however, are exactly the same as they would be if the rib were carrying a layer of rock 0.5**B** thick and of a width equal to **B**.

The following diagrams *Figs. 118* through *Fig. 121* illustrate various load assumptions. When comparing the diagrams the reader will notice that the sum of the active and passive force which acts on any one blocking point (total length of the arrow representing the pressure on the blocking point) is in every one of the diagrams, *Figs. 118 to 121*, the same. Since the external forces on the rib are the same, the internal stresses are identical.

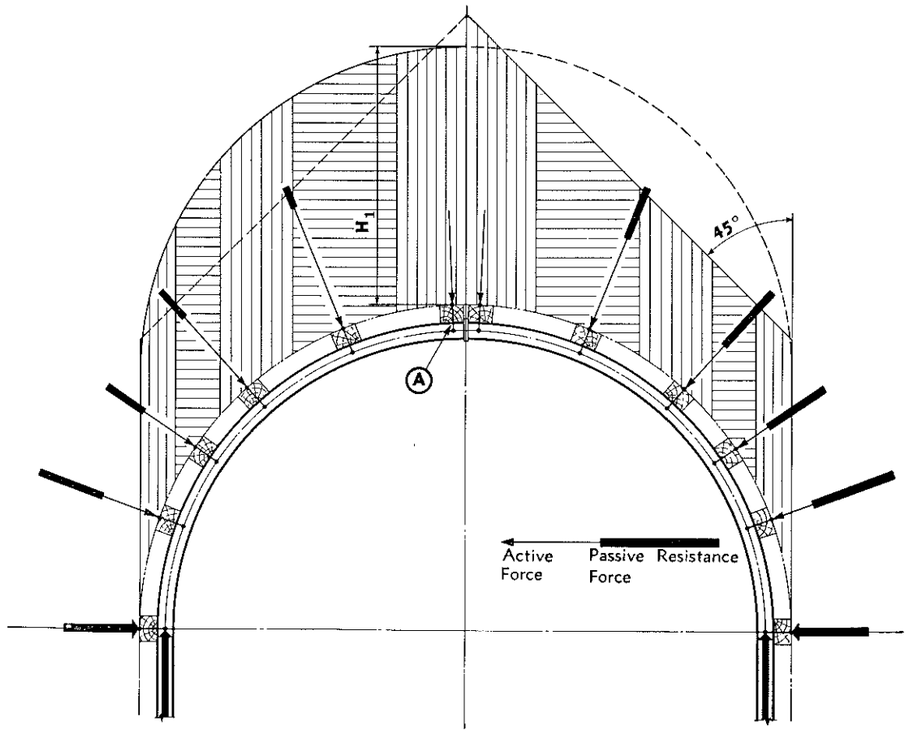


Fig. 118

Uniform load assumption **Triangular load assumption**

The total combined Active and Passive forces at corresponding blocking points are the same in each loading assumption.

Fig. 118 shows at the left side an assumed loading distribution in which a uniform vertical unit load acts on each foot of tunnel width. The shaded areas represent the rock which produces this loading. The height of this mass of rock is H_1. The arrows represent the external forces acting on the rib, the thin portions representing the active forces, the thick portions the passive resistance forces.

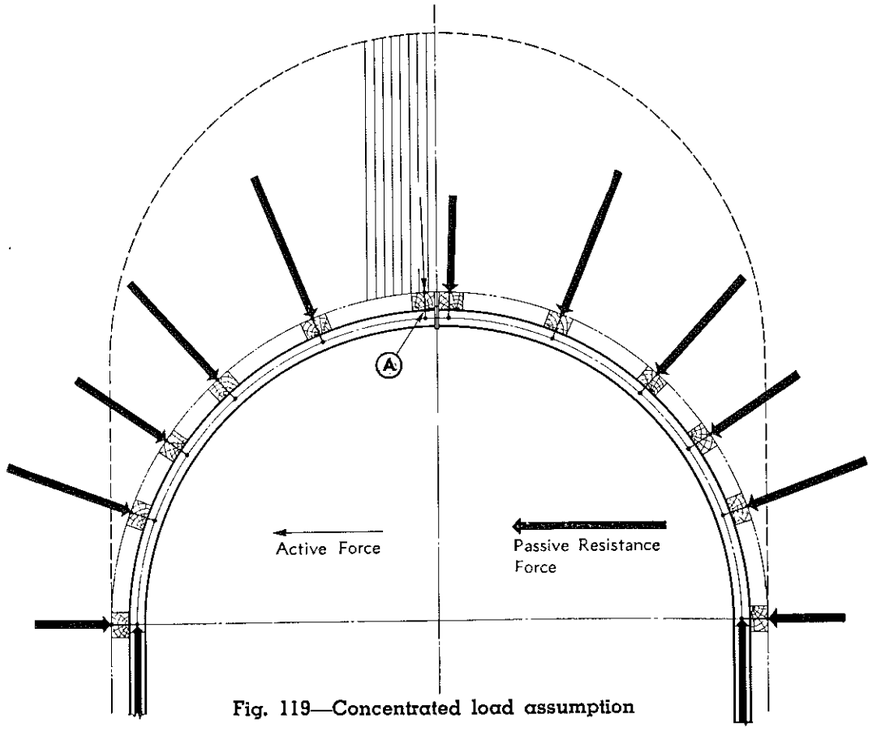


Fig. 119—Concentrated load assumption

A concentrated active load at one blocking point induces passive forces at all the other blocking points.

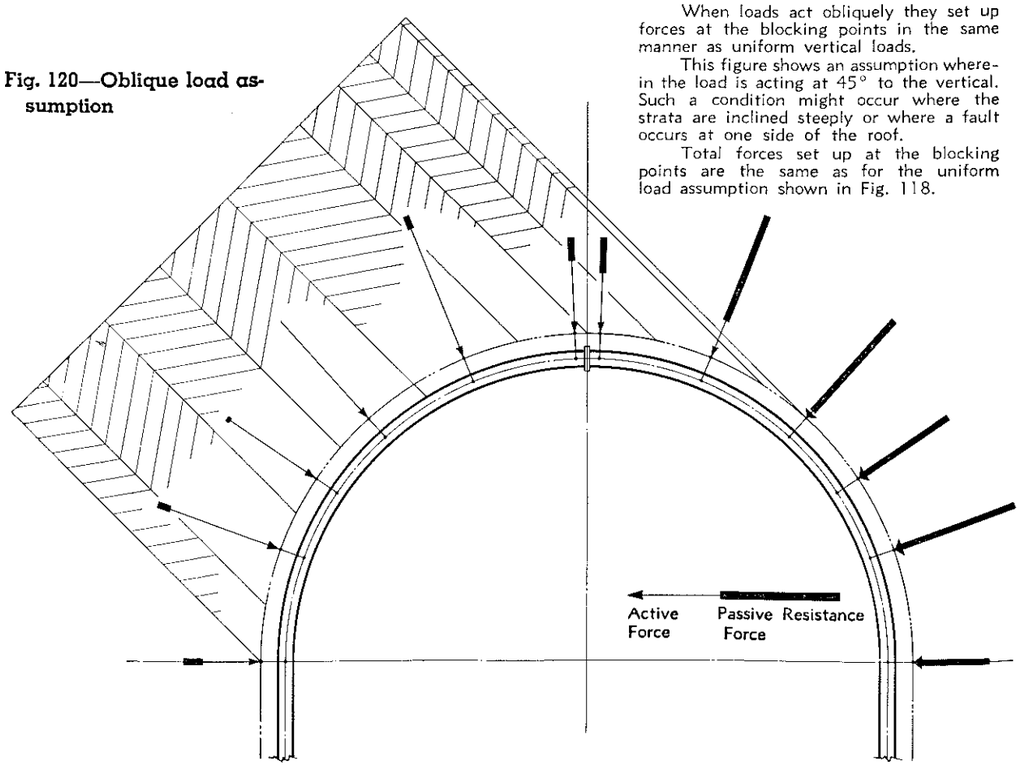
If the load at A is equal to the load at A in Fig. 118, forces at all the other blocking points are the same as in Fig. 118.

Fig. 120—Oblique load assumption

When loads act obliquely they set up forces at the blocking points in the same manner as uniform vertical loads.

This figure shows an assumption wherein the load is acting at 45° to the vertical. Such a condition might occur where the strata are inclined steeply or where a fault occurs at one side of the roof.

Total forces set up at the blocking points are the same as for the uniform load assumption shown in Fig. 118.



On the right side is shown a triangular loading assumption predicated on a 45° break line which imposes the same combined active and passive forces on the rib as the uniform load shown on the left hand side. This is evidenced by the arrows which are of the same overall length at corresponding points.

In *Fig. 119* it is assumed that blocking point A is acted upon by the same load that acts on point A in *Fig. 118* and that the other blocking points receive no active load at all. Yet the total force acting at any blocking point is the same as in *Fig. 118* and consequently the stress in the rib is the same.

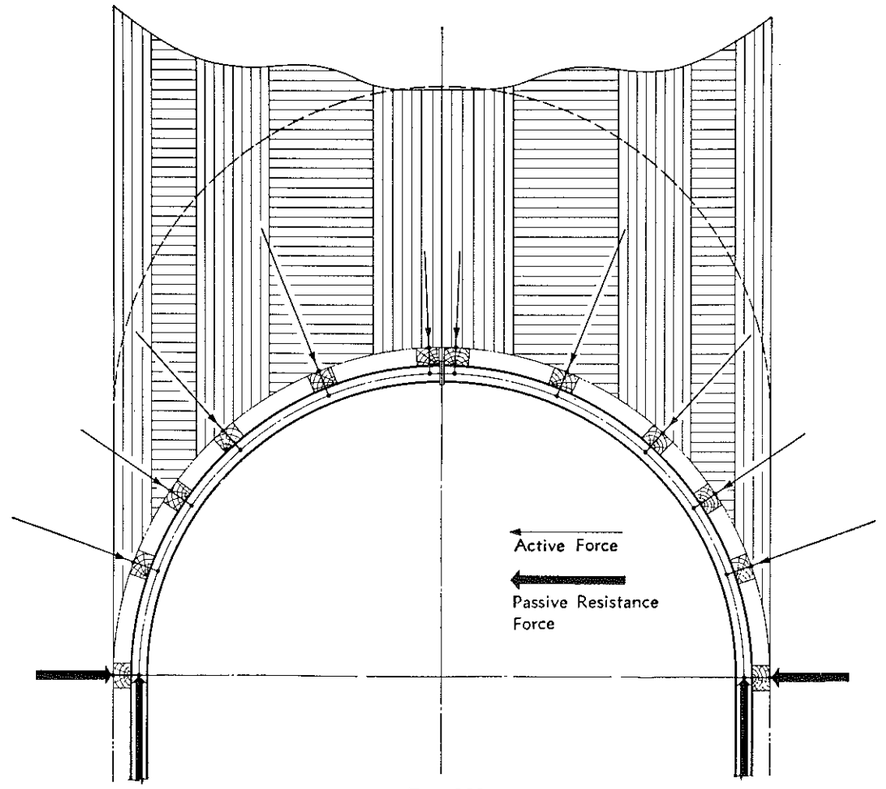


Fig. 121

Where it is possible to apply the exact amount of active load to each blocking point to balance all forces, the loading diagram could be assumed as in this figure.

All active forces are equal to the combined active and passive forces on the uniform load assumption, Fig. 118.

In *Fig. 120* it is assumed that the load acts at 45° to the vertical. Here also the total combined forces at corresponding blocking points are the same as those in *Fig. 118*.

Fig. 121 shows how much active vertical load can be applied at each blocking point without changing the external forces shown in *Fig. 118*.

From this comparison of the effect of various assumptions with regard to the load distribution of the forces and on the corresponding stresses in the rib, it is seen that

a uniform assumption is satisfactory. The principal source of error resides in the forecast of the rock behavior in different sections of the tunnel and evaluation of loads associated therewith. Compared to this error, any error associated with the assumptions regarding distribution of active loads is negligible.

ALLOWANCE FOR INITIAL DEFLECTION

Cause and effect of initial deflection

When rib sets are installed in a tunnel, they are blocked and wedged tight. This prestresses the rib, resulting in a certain amount of deflection of the rib, within the elastic range of the steel. The prestressed rib can be likened to a tightened spring, exerting an outward force against the rock through each block. This is often sufficient to prevent loosening of the rock, thereby forestalling any large increase of load.

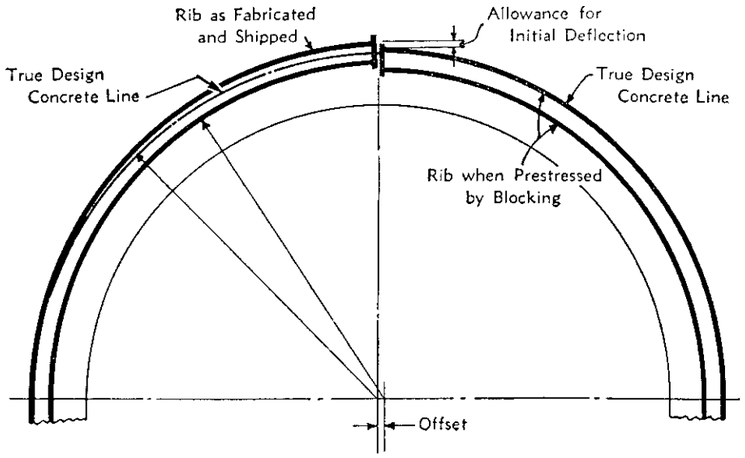


Fig. 122—Bending allowance for initial deflection

However, the initial deflection may show up as a subsidence of the crown joint, which may seriously encroach on the space provided for the concrete delivery pipe, and also on the design concrete.

Manufacturing allowance for initial deflection

To insure the proper space at the crown after wedging, the ribs are manufactured to a slightly larger radius than that indicated on the contract drawings. See *Fig. 122*. The center of curvature for each segment is also offset horizontally from the center shown on the contract drawings. Hence there will be a slight difference between the contract and shop drawings. The ribs will be above the true contour when placed, but will approach the true contour as wedges are tightened.

When the two halves of a rib set are placed together the inner edges of the crown butt joint will be in contact while the outer edges will be slightly separated. As the wedges are driven home, changing the shape of the rib in initial deflection, this joint tends to come into full bearing and the rib approaches the position called for by the plans.

RISE OR "KICK-UP" IN CROWN JOINT

The concrete lining in modern tunnel practice is usually placed by pumping or blowing the wet mix through a concrete delivery pipe placed at the crown. The pipe is commonly about 7 in. diameter over the couplings. A space of at least 9 in., preferably 10 in., must be allowed between the bottom of the crown joint and the forms or reinforcing rods for manipulating this pipe. In many smaller tunnels the design concrete is not thick enough to provide this space unless the crown joint of the rib is raised or "kicked up" or the ribs "set high." Setting the ribs above their designed position involves extra excavation and concrete in the tunnel arch. Space at the crown may be gained by making the inner surface of the concrete flat instead of curved as shown at A in *Fig. 123*. This has the effect of slightly decreasing the cross-sectional

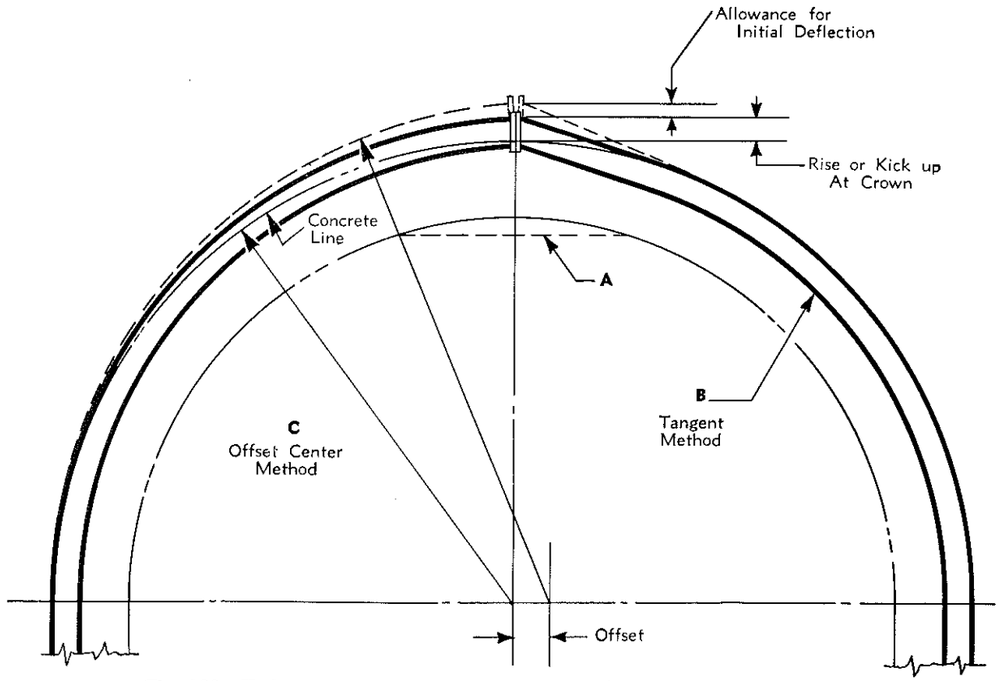


Fig. 123—Kick-up in crown joint to gain space for concrete delivery pipe

area of the tunnel, which is permissible in many cases. If reinforcing rods are being used near the inner face, however, it would not be helpful as the rods should follow the curve of the arch.

Methods of obtaining the rise of the crown point

The rise at the crown joint may be obtained (1) by leaving a short tangent on the rib as shown at **B** in *Fig. 123*, or (2) by using a larger radius swung from a center offset from the centerline of the tunnel as shown at **C** in *Fig. 123*. The rib dimensioned by this latter method departs from the design line by a small amount, increasing from zero departure at the spring line to the desired amount at the crown. In both methods the

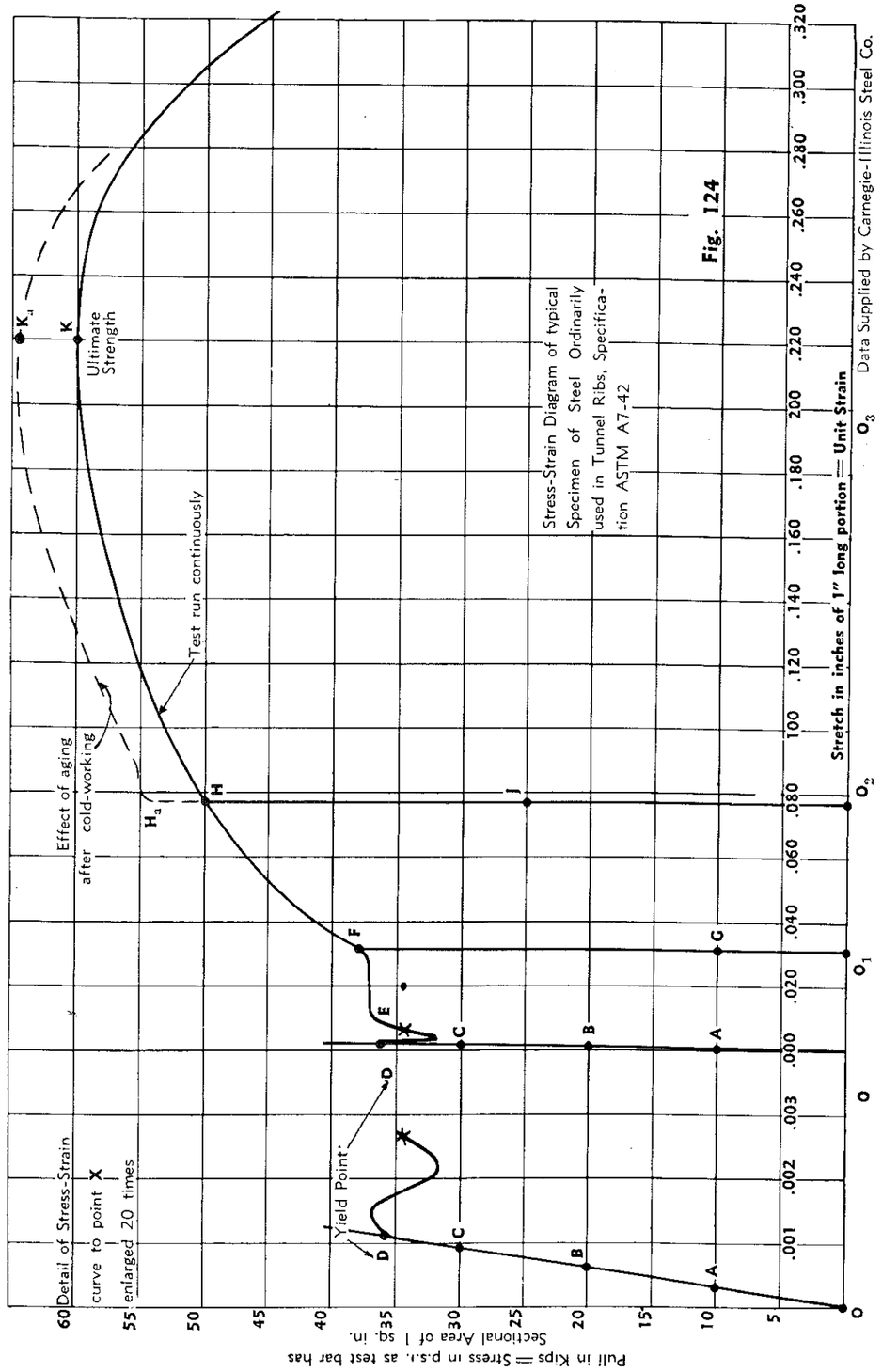


Fig. 124
Stress-Strain Diagram of typical Specimen of Steel Ordinarily used in Tunnel Ribs, Specification ASTM A7-42
Data Supplied by Carnegie-Illinois Steel Co.
Stretch in inches of 1" long portion = Unit Strain
Pull in Kips = Stress in p.s.i. as test bar has Sectional Area of 1 sq. in.
Ultimate Strength
Test run continuously
Effect of aging after cold-working
Detail of Stress-Strain curve to point X enlarged 20 times
Yield Point
O
O1
O2
O3

thrust induced in the rib by a given load, hence the stresses, are higher than if the crown joint is not kicked up. The offset center method (No. 2) is preferred to the tangent method (No. 1) as the values of thrust are considerably less than those due to using the tangent method. Inasmuch as any increase in thrust is associated with a decrease in capacity for a given rib section, the use of excessive rise at the crown should be avoided whenever possible. This is especially true for full circle ribs used in squeezing or swelling ground.

IMPROVEMENT OF STEEL BY COLD WORKING

Allowable stress in steel

The allowable stress in steel is commonly determined by the condition that the deformation produced by the stresses should be perfectly elastic. In other words the stresses should nowhere approach the yield point. When the stress slightly exceeds the yield point, it produces both elastic and very great plastic flow or permanent deformation. This permanent deformation at the yield point is of the order of 30 times the simultaneous elastic deformation.

Yield point of ordinary structural steel

To demonstrate the meaning of the term *Yield Point,* a bar of ordinary structural grade steel, ASTM A7-42, as commonly used in tunnel ribs, whose cross-sectional area is exactly one square inch, is placed in the jaws of a hydraulic testing machine having a pulling capacity of 40 tons or more. Let us consider a portion one (1) inch long and measure the change of length in this one inch portion by means of a recording extensometer.

As a pull is applied the steel bar will stretch slightly. If 10,000-lb. pull is applied, this one inch section will stretch .0003 in., the new length then being 1.0003 in. This is represented by Point **A** on the stress-strain diagram shown in *Fig. 124.* Release the pull and the bar will return to its original length 1.0000 in., Point **O.** This is "elastic deformation."

Apply a 20,000-lb. pull and it will stretch to 1.0006 in., Point **B,** and when the pull is released, it will again return to its original length 1.0000 in. The same phenomenon takes place at 30,000-lb. pull (Point **C**). The stretch is still within the range of elastic deformation.

Now we start again increasing the pull slowly and uniformly, watching the hydraulic pressure gauge which registers the pull. As the pull becomes 10,000 lbs. (Point **A**), 20,000 lbs. (Point **B**), etc., it will be observed that the length has again become 1.0003 in., 1.0006 in., etc., as before. In other words, the stretch is proportional to the pull.

However, at some pull above 33,000 lbs., (for the steel under consideration, at 36,000 lbs. Point **D**,) the bar will stretch quite rapidly, so rapidly in fact that the hydraulic pressure gauge will show a slight temporary drop in the pull. When the pull again becomes 36,000 lbs., Point **E**, the new length will be 1.0140 in. A slight increase of pull to 38,000 lbs. increases the length to 1.0320 in., Point **F**. Then the pull is entirely released and the bar contracts, not to its original length, but to a permanent new length, 1.0308 in., Point O_1. The difference between the original length of 1.0000 in. and its new length represents a permanent set.

Point **D**, at which this phenomenon occurs is called the *Yield Point,* and while not strictly correct, the 36,000 lb. pull is called the *Elastic Limit.* It is observed that

the permanent deformation is roughly 30 times the elastic deformation at a pull slightly lower than the elastic limit.

Elastic range of cold worked steel

Now apply a 10,000 pull again and the bar stretches the same amount as initially for this pull, the new length being 1.0311 in., Point **G**. Release the pressure, and it returns to the deformed length of 1.0308 in., Point O_1, indicating elastic behavior. Repeat for any pull up to 38,000 lbs., and the same thing happens. Thus it is seen that the steel now has a new elastic limit of 38,000 lbs.

Then run the pull steadily up to 50,000 lbs., Point **H**. As the pull passes 38,000 lbs., it will be observed that the bar stretches faster than it did below 38,000 lbs., but there is no sudden yielding such as happened the first time the pull reached 36,000 lbs. The steel no longer has a well defined yield point. At 38,000 lb. the stretch imperceptibly changed from elastic deformation to plastic flow. At Point **H** the length has now become 1.0780 in. Release the pull entirely and the piece will not return to its original deformed length but to a new one 1.0765 in., Point O_2.

Any pull up to 50,000 lbs. may now be applied, Point **J**, and released, and the bar will return to 1.0765 in. Thus, the elastic limit has been increased from 38,000 lbs. to 50,000 lbs., and the yield point has been practically eliminated. Lines O_1—**F**, and O_2—**H**, plotted for any number of values of points **G** and **J** will be roughly parallel to the original elastic line **O—D**. This statement applies to every value of pull represented by a point on curve **F—K**. However, if the pull is raised to 60,000 lbs., point **K**, the bar stretches rapidly. The pull of 60,000 lbs. cannot be maintained, as indicated by a drop in pressure on the hydraulic gauge. Rupture soon occurs, and at some pull below the 60,000 lbs. This process is associated with what is known as *necking down;* as the piece stretches, its cross-sectional area decreases at some point, so that it no longer has a square inch of area to resist the pull of 60,000 lbs. The 60,000 lbs. pull, Point **K**, is called the *Ultimate Strength*. The region represented by the stress-strain curve between **D** and **K** is known as the *Region of Plastic Deformation* of steel.

Influence of cold working on elastic limit

On account of the existence of the plastic range represented by the section **D—K** of the stress-strain curve in *Fig. 124*, steel can be worked in a cold state. One of the most outstanding applications of the cold working processes to industrial purposes is the automobile industry. Another is the manufacturing of ribs for tunnel supports.

From *Fig. 124* it is obvious that cold working raises the elastic limit of ordinary structural steel from a minimum of 33,000 lbs. per sq. in. to a figure close to its ultimate strength which ranges between 60,000 and 72,000 psi; also that the yield point is practically eliminated. (See note below.) The elastic limit after cold working varies with the amount of plastic deformation that occurred in the process. The elastic limit for any cold worked rib may be roughly determined on the basis of the data shown in *Fig. 124* by calculating the change in any one-inch element of the rib required to give the steel beam its new shape. By plotting the change on the base line a point O_r is obtained. Then a line O_r—D_r is drawn parallel to **O—D**. The ordinate of the intersection with the curve **F—K** would represent the new elastic limit of the cold worked rib.

By annealing cold worked steel, the benefits derived from cold working are lost because the elastic limit returns to its original value, 36,000 lbs., whereupon the steel reacquires its original yield point. Hot formed steel retains both its original yield point and elastic limit, and may even drop below, depending on the amount and duration of heat.

Fig. 125—Failure in unworked portions of steel ribs.

Failure in the straight legs of ribs due to overload. The metal in the straight leg has not been cold worked, hence retains its yield point and its original elastic limit. The steel in the curved arch portion of the ribs has been cold worked and therefore stands much higher stress before beginning to fail.

Metropolitan Water District of Southern California

LOS ANGELES-COLORADO RIVER AQUEDUCT, WIDE CANYON TUNNEL NO. 1

Contractor: Force Account

Tunnel dimensions: driven bore, 18′-2″ wide by 17′-10″ high, horseshoe; finished bore, 16′-0″ by 16′-0″, horseshoe.

Support: continuous ribs, 6″ I-beams, 12.5 lbs. shown in this photograph at about 1 ft. centers.

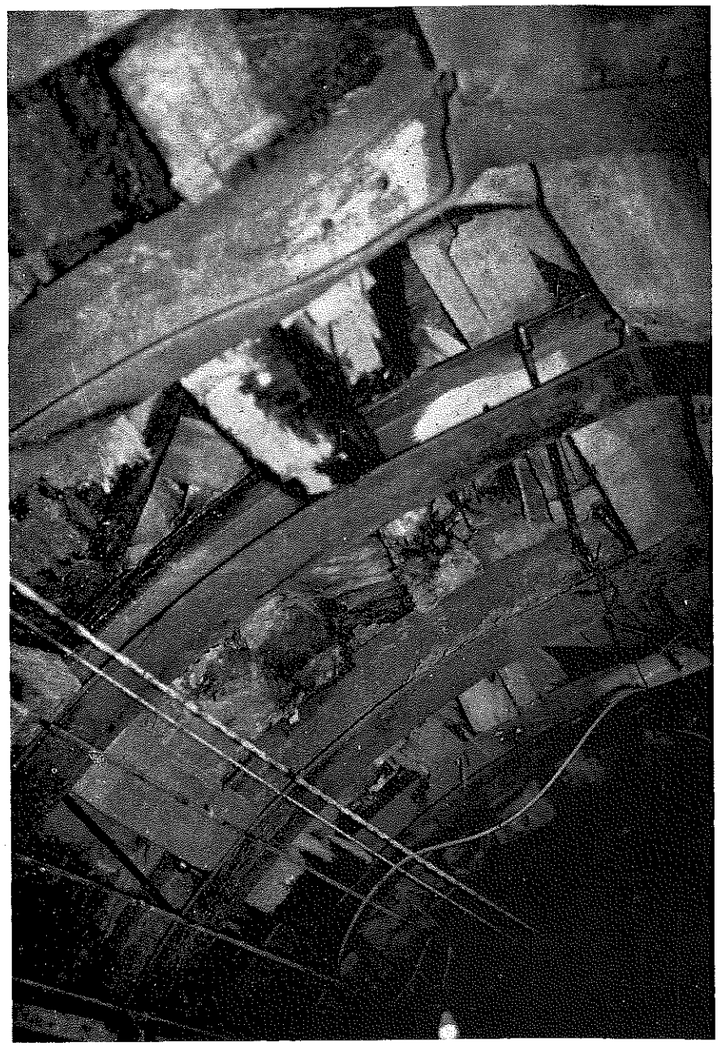


Fig. 126—Failure of a rib in the straight tangent portion adjacent to the crown joint.

The metal in the straight portion was not cold worked. When stresses passed the normal yield point of the steel, failure occurred but did not extend into the cold worked curved portion of the rib. The crushed blocking in the upper left gives some idea of the pressures which acted on the ribs.

City of Los Angeles, Bureau of Water & Power
MONO BASIN PROJECT, MONO CRATERS TUNNEL
near Leevining, California

Tunnel dimensions: driven bore, 11′-9″ wide by 12′-3½″ high, horseshoe; finished diameter, 9′-7½″.

Support: continuous ribs, 6″ H-beams, 20 lbs., spacing as shown here, 2 ft.

Cold worked versus hot worked ribs

In tunnel ribs with straight legs the arch portion can be formed cold or hot, whereas the straight unworked leg retains its original properties. A rib will sustain increasingly heavy loading just as long as all portions of the rib retain their shape within the limits of elastic deformation. The highest stresses in tunnel ribs develop in the flanges. If the stress in some small portion of a hot-worked rib flange reaches the yield point, this portion changes shape very seriously because the deformation at the yield point can be about 30 times greater than the elastic deformation. Once a part of the flange is badly deformed, the metal in this small part is no longer in the right place to carry its full share of the load. The metal adjacent is thereby stressed beyond the yield point, causing it to change shape, and complete failure may result.

In the cold-worked portion of the rib the stress required to start a local change of shape is considerably higher than in the unworked leg. And when distortion does start it requires constantly increasing load to produce further change of shape, as the steel no longer has a yield point worth considering.

The forces acting on the curved arch section of the rib are usually much greater than those acting on the straight leg. Yet it has been observed that failures of overloaded cold-worked ribs invariably occur in the unworked straight portions of the rib as shown in *Figs. 125* and *126*. This is due to the fact that cold-worked steel has a reserve of strength which is not possessed by unworked steel nor by ribs formed hot.

If the rib consists of a cold-worked curved portion and straight unworked portions, the strength of the rib is limited by the unworked portions. The extra strength of the curved portion merely increases its factor of safety. On the other hand, when the ribs are bent from end to end, the designer is justified in raising his allowable fibre stress. Such would be the case for full circle ribs used to hold a squeeze.

Note: The stress-strain curve in *Fig. 124* is a plot of the readings observed on a single test specimen. The ASTM A7-42 specification requires a minimum yield point of 33,000 psi., but according to the results of mill tests the yield point usually ranges between 36,000 and 45,000 psi. The specification also requires a minimum ultimate strength of 60,000 psi., whereas it usually ranges between 60,000 and 72,000 psi. This curve then represents more nearly the minimum. But no matter what individual specimen is used, the shape of the curve is about the same as the one shown in *Fig. 124*, and generally the curve will lie above it.

Effect of ageing on cold worked steel

The stress-strain curve and the effect of cold working discussed in the foregoing paragraphs are typical when the test is run continuously. If, however, the specimen is stressed to some point, as **H** in *Fig. 124*, in the region of plastic deformation, the pull removed for several weeks, and then the test resumed, the steel acquires a slight yield but at a higher value $\mathbf{H}_a$ of pull than was applied at **H**. The ultimate strength $\mathbf{K}_a$ is also increased. This ageing effect is shown by the dotted line in *Fig. 124*. The closer point **H** is to **K**, the greater is the distance between curves **H-K** and $\mathbf{H}_a$-$\mathbf{K}_a$ after ageing.

As applied to cold-worked ribs, usually a period of several weeks elapses between the time of forming and the time when the ultimate load is imposed on the ribs. Thus the ageing effect has further strengthened the cold-worked rib.

Fig. 127—Typically irregular blocking of ribs in a rock tunnel.

Curved steel ribs are commonly blocked to the rock at points where overbreak is favorable. It is not necessary to install blocks in any fixed pattern except that spacing should not exceed a designed maximum distance. Each butt joint should be blocked, and blocks also tightly set at the springline.

In this large tunnel all lagging which could be removed has been taken down just prior to concreting so the concrete can flow to the rock surfaces through and around the blocking.

Hydroelectric Power Commission of Ontario (Canada)
Niagara Falls, Ontario

INVERTED SIPHON FOR SIR ADAM BECK GENERATING STATION NO. 2

Contractors: Part by Perini Corporation, Part by Rayner Atlas Construction Co.

Tunnel Dimensions: Driven diameter 50'-0" min. fin. dia. 45'-0".

Support: Rib and wall plate 8" x 8" ribs 4'-0" c/c, wall plates double 10" I-beams set on timber posts. Short posts constituted some hazard due to narrow shelf for footing.

CHAPTER 10

FORCES AND STRESSES IN BLOCKED RIBS

BLOCKING POINTS AND FORCES

Spacing between blocking points

If forces are applied to a rib only at widely spaced points, the rib is acted on not only by axial thrust but also by bending moments. Bending moments, and the stresses arising therefrom, cause a reduction in the load carrying capacity of the rib. In a given rib the bending moment depends on the spacing of the blocks and varies as the square of the spacing. Therefore the spacing between blocks deserves careful attention.

The first step in determining the maximum stresses in a rib is to make a layout for the blocking points for one half of a rib set. One blocking point is required close to the crown joint and one at the spring line or at the junction between arch and leg if the leg is battered. The spacing of the intermediate blocking points depends on the condition of the rock. The sounder the rock, the greater the spacing can be. On account of the erratic irregularities of the rock surface, a blocking point can be established only where a suitable rock surface is available. Hence the spacing of the blocks as actually installed in the tunnel will deviate widely from the average. *Fig. 127* and *Fig. 117*, page 193, show typical irregularities of spacing.

The layout of the intermediate blocking points is governed by the following considerations.

a. Each block installed is associated with a cost for labor and material. The more blocks required, the higher the cost of the blocking operation. While it is natural for the builder to block all rock that looks dangerous, there is a tendency to leave out blocks that should be placed against sound rock for support of the rib.

b. For a given shape of rib and a given rock load the bending moments in the rib increase with increased space between blocks because the rise of the arc between blocking points increases. The rise of the arc, hence the bending moment, increases as previously stated, as the square of the distance between blocking points. The import-

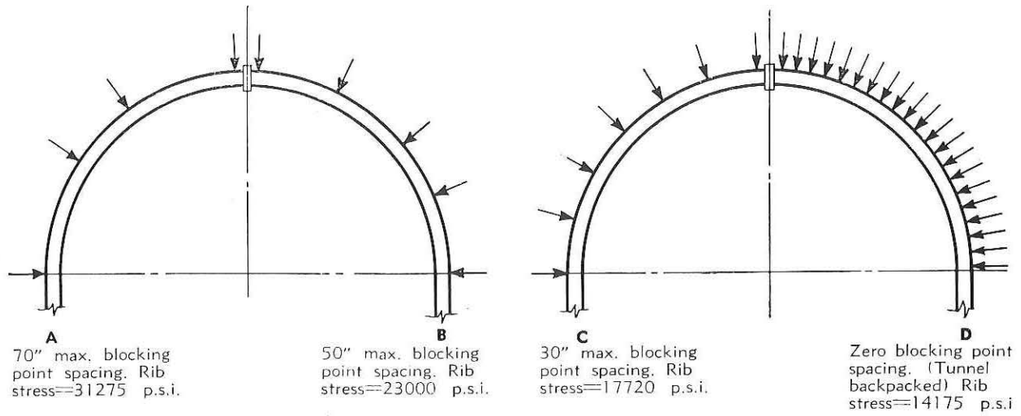

A
70" max. blocking point spacing. Rib stress=31275 p.s.i.
B
50" max. blocking point spacing. Rib stress=23000 p.s.i.
C
30" max. blocking point spacing. Rib stress=17720 p.s.i.
D
Zero blocking point spacing. (Tunnel backpacked) Rib stress=14175 p.s.i

Fig. 128—Effect on rib stress of changes in blocking point spacing

ance of the influence of the spacing on the total stress in the rib is illustrated in *Fig. 128*. The computation of stress in the rib **B** can be found in *Example 1* on page 219. It was computed on the basis of 50-in maximum blocking point spacing. At **A** is shown the effect of increasing the maximum spacing to 70 in. **C** shows the effect of decreasing the maximum spacing to 30 in. and **D** shows the effect of reducing the spacing to zero. The difference between the fibre stress at **D** and any of the others represents the reduction of the rib capacity due to bending.

The rib must be dimensioned on the basis of the most unfavorable conditions which may prevail in the field. Therefore the design is based on the maximum spacing between blocks which is tolerable in the tunnel. The blocking point spacing used in *Tables 1* and *2* in the appendix may be used as a guide in selecting the spacing on which the design is to be based.

Assignment of rock loads to blocks

In *Figs. 118* to *121*, the shaded areas represent that part of the rock which is assumed to be acting on the respective blocks. It constitutes the rock load. The term *uniform load assumption* refers to a rock load the height of which is uniform over the full width of the tunnel. The upper boundary of such a rock load has the same shape as the roof of the tunnel, as shown on the left hand side of *Fig. 118*.

Blocking points defined

Each block transfers one part of the total rock load onto the rib. The method of allocating the rock load to the various blocks is illustrated by *Fig. 134* and will be

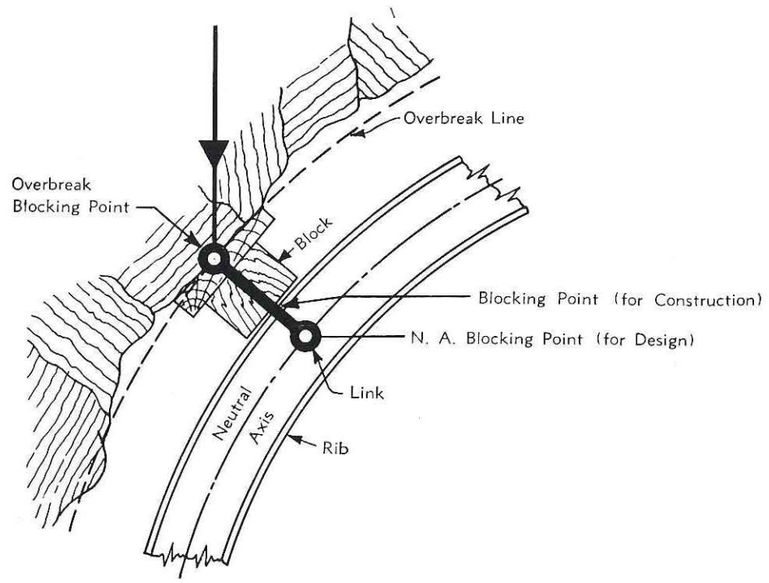


Fig. 129—Transfer of force from rock to rib

Every block is considered to be a link as it has very little resistance to shear deformation. Forces are considered to be applied in a direction normal to the tangent of the rib at the neutral axis.

explained in detail in a following paragraph. The rock acts on the block at the overbreak line. The center of contact between the block and the rock surface will be called the *overbreak blocking point*.

The blocks cannot transmit tangential forces because they have very little resistance to shear deformation. Hence they act for all intents and purposes like pin-connected links between the rock and the rib. They are oriented normal to a line tangent to the rib at their points of contact as shown in *Fig. 129*. The inner pin, assumed to be on the neutral axis of the rib, will be referred to as the *neutral axis blocking point* for computation purposes. The point of contact between block and rib is referred to as the *blocking point* for construction purposes.

Action of forces at overbreak blocking points

At the moment the rib is installed there is no active rock load or force; otherwise the rock would previously have fallen. The rock load is carried by dome or arch action of the rock itself, or by crown bars or temporary posts. Wedging prestresses the rib, similar to tightening a spring. Because of the initial elastic deflection associated with prestressing, the rib exerts an active force against the rock at each overbreak blocking point through the block.

This active force is represented by F_r in *Fig. 130 a*. In this figure the dash-dot lines represent the overbreak line and the small circles the overbreak blocking point. The rock resists by mobilizing a passive force F_p, equal and opposite, thereby holding the point in equilibrium. In the various diagrams in this book, force arising out of active cause is shown as a thin arrow, whereas passive force is shown as a thick arrow, as in *Fig. 130*.

As the dome action of the rock is superseded by arch action, as the temporary posts are removed or the crown bars freed, or as succeeding shots loosen the rock,

The active force F_r exerted by the prestressed rib is met by an equal and opposite passive force F_p induced in the rock.

The rock begins to develop weight. It exerts an active vertical force **W** on to the block. The resultant of the two active forces **W** and F_r induces an equal and opposite passive force F_p.

The active rock force **W** increases. Since the resultant of **W** and F_r induces an equal and opposite passive force, F_p, the magnitude of F_p changes and its direction passes through the horizontal.

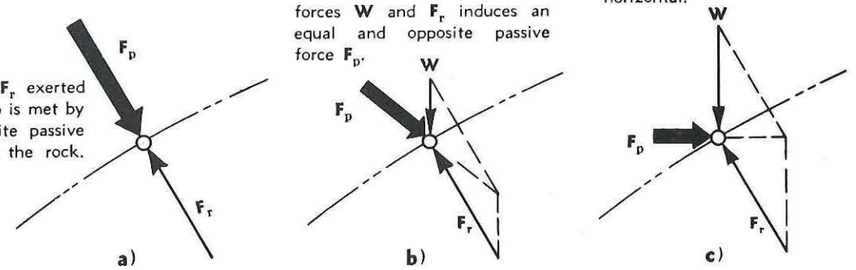
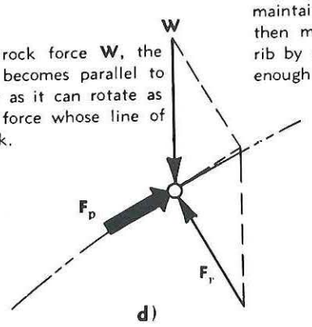


Fig. 130—Forces acting at overbreak blocking point

the rock contiguous to the rib under consideration begins to "develop weight." In other words, a vertical force **W** now acts on the overbreak blocking point as shown at b. Since force F_p is the balancing force, purely passive, its magnitude and direction can change to suit varying conditions. Therefore, as **W** develops, the magnitude of F_p changes and its line of action rotates (counter-clockwise in this example) until it

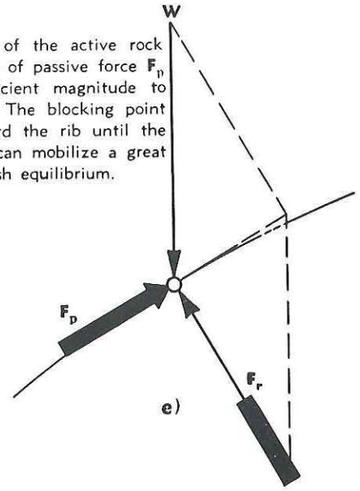
balances forces W and F_r. As the active force W increases, the direction of F_p continues to change, passing through the horizontal as at c, until it is acting along a line normal to the rib force F_r as at d. This line of action is tangent to the overbreak line and is considered to be the limit to which F_p can rotate because equilibrium requires that the line of action shall lie within the rock and pass through the blocking point.

With increasing active rock force W, the direction of F_p ultimately becomes parallel to the tangent. This is as far as it can rotate as the rock cannot supply a force whose line of action lies outside the rock.



d)

With further increases of the active rock force W and the direction of passive force F_p fixed, F_r is not of sufficient magnitude to maintain the equilibrium. The blocking point then moves slightly toward the rib until the rib by deflecting slightly can mobilize a great enough Force F_r to establish equilibrium.



e)

Fig. 130—(Continued from page 209)

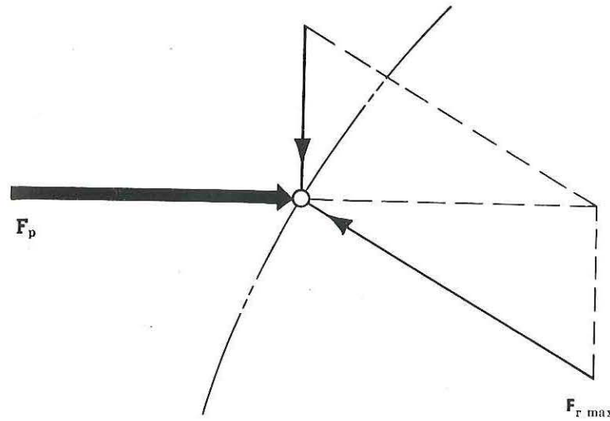
Any further increase in the active vertical force W changes the character of the force F_r exerted by the rib from active to passive. The rib no longer is tending to displace the rock, but the rock is now tending to deform the rib, which must mobilize a greater force to maintain equilibrium as shown in *Fig. 130 e*. For the rib to exert a greater force it must deflect somewhat which permits the blocking point to move slightly. This movement, though measured in small fractions of an inch, may permit the rock to loosen and thereby further increase the active force W. In practice the ribs should be kept wedged so tightly that the contemplated active forces W can never cause the force F_r to become passive. By so doing loads W can be minimized.

The designer assigns a value to the force W derived from his assumption of the active vertical loads. The rib, then, must be so designed that it can supply the radial force F_r in a magnitude sufficient to maintain equilibrium. The maximum fibre stress in the rib must not exceed the fibre stress considered to be safe.

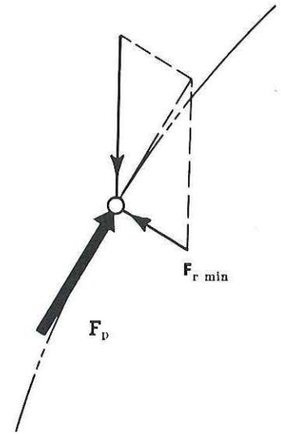
Assumption with respect to action of the load

In *Fig. 118*, each block is assumed to carry a rectangular prism of rock bounded by four vertical planes. The longitudinal planes pass through the mid-points between the blocks; the transverse planes pass through the mid-points between ribs. The top boundary is assumed to be unattached to the rock above. This prism may be referred to as the *active prism*.

A further assumption must be made regarding the direction of the passive force F_p by which the surrounding rock reacts onto the active prism, because this direction determines the magnitude of the radial force F_r required of the rib for a given load W. An upper limiting value $F_{r\,max}$ of the required rib force is obtained when assuming that the vertical sides of the active prism are frictionless. On this assumption the passive force F_p acts in a horizontal direction. The magnitude $F_{r\,max}$ of the radial force is obtained by means of the parallelogram of forces, *Fig. 130 f*.



f)



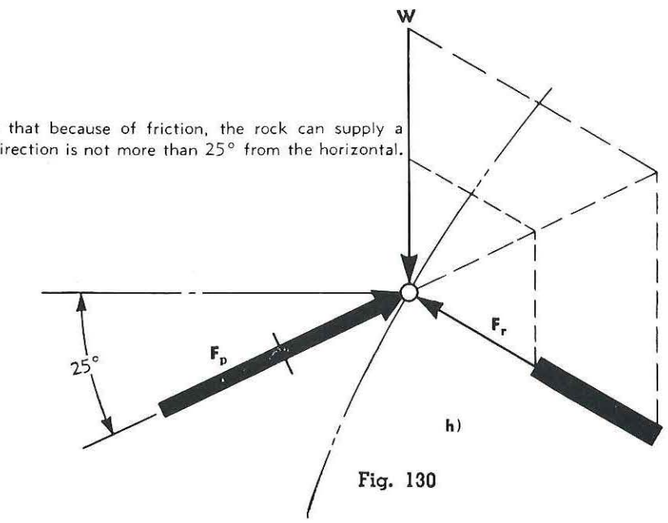
g)

Fig. 130—(Continued from page 210)

On the other hand, a lower limiting value $F_{r\,min}$ is obtained when assuming that F_p reacts in a direction parallel to a line tangent to the overbreak line at the blocking point as shown in *Fig. 130 g*. The corresponding required radial force, $F_{r\,min}$ is smaller than $F_{r\,max}$ as is readily seen by comparing *Figs. 130 f* and *g*.

The values $F_{r\,max}$ can be much greater than the values $F_{r\,min}$. The real value of F_r is intermediate between $F_{r\,min}$ and $F_{r\,max}$. It depends on the orientation of the real joints, and may vary from place to place. However, considering the conceivable orientation of the real joints, and the fact that real joints are not planes, and that joints are the seat of friction and interlock, it is assumed that the adjacent rock can react on the active prism with a force applied at some angle, the vertical compon-

It is assumed that because of friction, the rock can supply a force F_p whose direction is not more than 25° from the horizontal.



h)

Fig. 130

ent of which force is equal to the friction. This angle is arbitrarily taken 25° from the horizontal as in *Fig. 130 h.* This means that as the active vertical force **W** increases from zero, while the active rib force $\mathbf{F}_r$ mobilized by prestressing, remains constant, the direction of the passive force $\mathbf{F}_p$ can rotate till it reaches the tangent line or the arbitrary limit of 25° below the horizontal. Beyond this point a further increase in **W** brings about a conspicuous change in the character of the force $\mathbf{F}_r$. The active force $\mathbf{F}_r$, which arises out of the elastic deformation of the rib caused by the initial wedging, is no longer of sufficient magnitude to maintain equilibrium. The blocking point moves slightly toward the tunnel thereby deforming the rib slightly. (The spring is wound tighter.) This deformation enables the rib to mobilize a passive force to add to the initial active force to restore equilibrium. This increase in the force $\mathbf{F}_r$ is indicated by the heavy portion of the arrow $\mathbf{F}_r$ in *Fig. 130 h.*

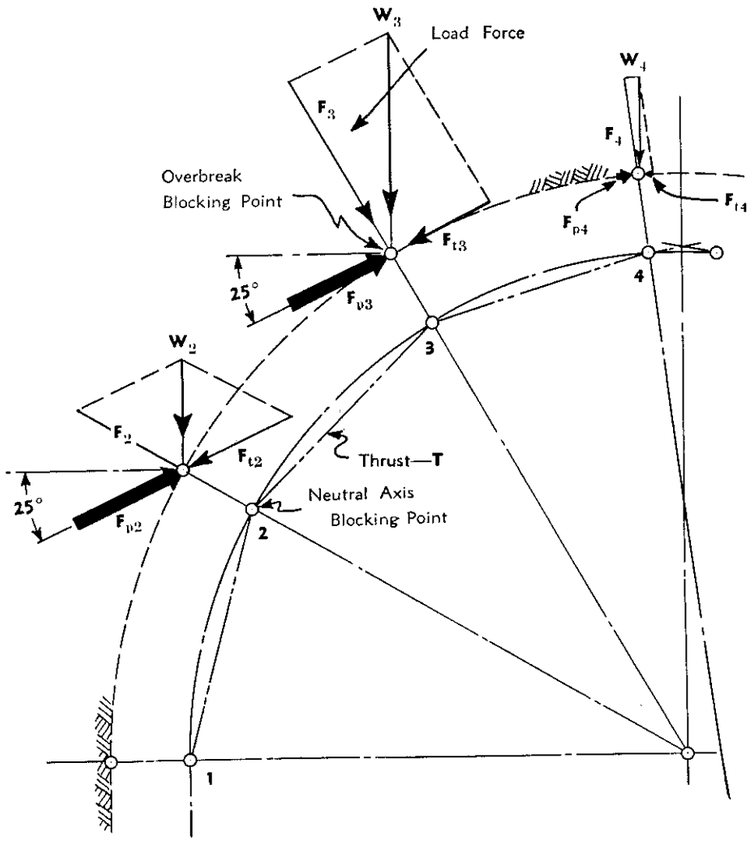


Fig. 131—Resolution of forces at overbreak blocking points

The assumed active vertical force **W** at each blocking point is resolved into two components, a radial load force **F** and a component force $\mathbf{F}_t$, acting at 25° to the horizontal or at the slope angle of the tangent, whichever is smaller. The force $\mathbf{F}_t$ induces an equal and opposite passive force $\mathbf{F}_p$ in the rock which can be of any required magnitude. Hence $\mathbf{F}_t$ is disregarded. The radial load force **F** either induces or opposes the rib force ($\mathbf{F}_r$ in **Fig. 130 a**) exerted through the block.

In semi-circular arches with the load assigned to the blocking points as in *Fig. 118,* the slope angle of the passive force $\mathbf{F}_p$ nowhere exceeds 25°.

Transfer of forces to the rib

Fig. 131 shows the neutral axis of one half of a semi-circular rib with 4 blocking points, 1 to 4, and the center lines of the "pin-connected links" which transfer the rock loads onto the ribs. Blocks 2 to 4 are acted upon by vertical rock loads **W.** Since a pin-connected link can transfer force only in the direction normal to a line tangent to the rib at the blocking point, each force **W** is resolved into two components. One component **F,** called the *load force,* is normal to the rib and acts on the rib through the "link," at the neutral axis blocking point. To fulfill its function of supporting the tunnel roof, the rib must be capable of supplying an opposing force of equal or greater magnitude. The other component $\mathbf{F}_t$ is tangential, if the direction of the tangent is 25° or less from the horizontal; otherwise it is at 25°. The rock mobilizes a passive force $\mathbf{F}_p$ equal and opposite which neutralizes $\mathbf{F}_t$. Since the available passive resistance of the rock is far greater than the greatest value $\mathbf{F}_t$ can possibly assume, this component $\mathbf{F}_t$ can safely be disregarded.

Forces acting at neutral axis blocking points

In order to compute the magnitude of the forces which act in the rib it is tempor-

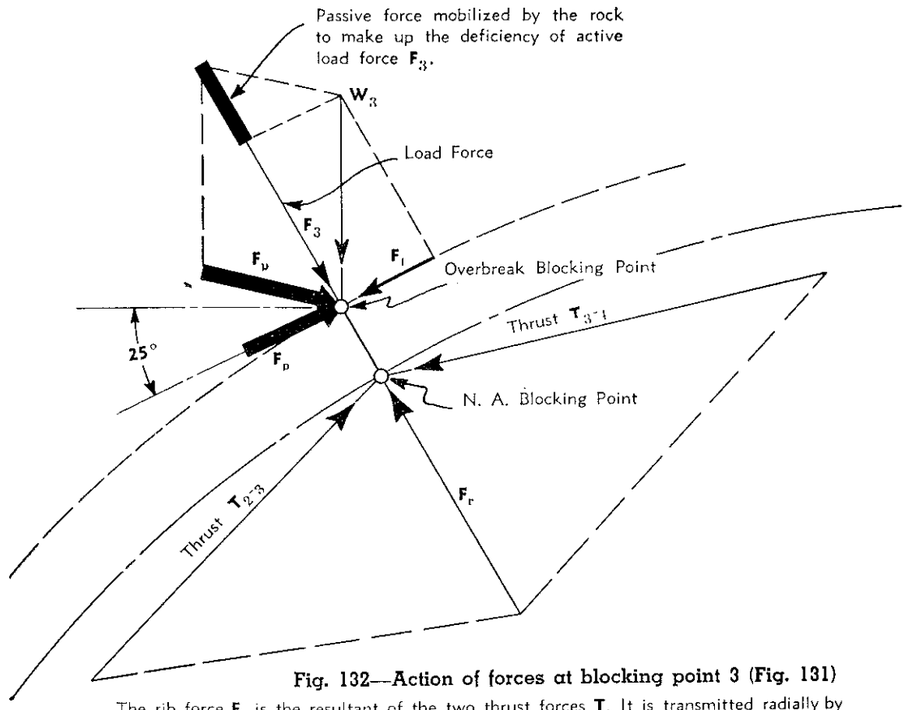


Fig. 132—Action of forces at blocking point 3 (Fig. 131)

The rib force $\mathbf{F}_r$ is the resultant of the two thrust forces **T.** It is transmitted radially by the block to the rock where it is met by the active load force **F.** This in turn is the radial component of the active vertical load force **W.** If **F** is less than the rib force $\mathbf{F}_r$, the rock mobilizes additional passive force by changing the direction and magnitude of passive force $\mathbf{F}_p$ so that the resultant of $\mathbf{F}_p$ and **W** is equal to $\mathbf{F}_r$.

arily assumed that the rib consists of pin-connected segments, 1-2, 2-3, etc. On this assumption the bending moments at blocking points are zero and the thrust acts in the direction of the straight line interconnecting adjacent blocking points. Each neutral axis blocking point is therefore acted upon by three forces; a normal load force **F** coming from the rock through the block and two thrust forces **T** coming from adjacent blocking points as shown in *Fig. 132.*

Every blocking point is in a state of equilibrium. Yet it can readily be seen that this equilibrium cannot exist unless force $\mathbf{F}_3$ is equal and opposite to the resultant of thrusts $\mathbf{T}_{2\text{-}3}$ and $\mathbf{T}_{3\text{-}4}$. This resultant is the force exerted by the rib, force $\mathbf{F}_r$ in *Fig. 130.* If force $\mathbf{F}_3$ were removed, the point 3 would tend to move out. Yet point 3 remains where it is because the rock prevents such movement by mobilizing a force of passive resistance at the overbreak blocking point as in *Fig. 130.* Hence if the load force $\mathbf{F}_3$ resulting from the active load $\mathbf{W}_3$ is insufficient to establish equilibrium, passive resistance makes up the deficiency as shown in *Fig. 132.*

As a general rule, in any rib loaded at a series of blocking points, maximum thrust will be induced by the load force **F** at some one point. The rib will be proportioned for this maximum. Because of this, the forces $\mathbf{F}_r$ exerted by the rib at all other points will be greater than the active load forces **F** at these points. Hence the load force **F** will be entirely active at only one point and will be active and passive or entirely passive at all other points. In other words, regardless of whether the load on the tunnel roof is assumed to be uniform or concentrated, the forces are determined by one concentrated load.

The application of this general principle to the computation of the thrust in ribs will be explained in the following paragraphs.

METHOD OF FORCE AND STRESS COMPUTATION

Polygon of forces

Each blocking point is held in equilibrium by the forces acting on it in exactly the same manner as a panel point in a truss. Graphical determination of the *thrust* in the same manner as in a truss design is a quick and easy method and, considering the uncertainties involved in the loading assumption, is accurate enough.

The engineer will make a *loading diagram* and then a *force polygon,* calling in whatever passive resistance force he needs at each blocking point to establish equilibrium and close the polygon.

Thrust

Thrust is the force that sets up all stresses in the rib.

1. Thrust, acting uniformly over the cross section of the rib, produces uniformly distributed compressive stress.
2. The eccentricity of the thrust with reference to the rise of the arc between blocking points produces bending moment, hence flexure stresses.

Values of thrust may be scaled from the rays of the force polygon.

Bending moments in the arch

The thrusts represented by the rays of the polygon act in the direction of a straight line connecting two adjacent blocking points. If it were possible to predict the shape of the rock overbreak, ribs could be made up of sections straight between blocking

points which, in turn, would eliminate bending stress. However, unavoidable irregularities in the overbreak make it impracticable to block to any pre-determined pattern. This is one of the sources of expense when using timber sets where it is mandatory to block at the panel points.

Steel ribs are curved to permit blocking at the most favorable points. Because of the curvature between blocking points, however, the ribs are subject to bending. This is not a serious matter as the amount of bending stress is relatively small with normal spacing of blocks. (See *Table 1* on page 238 of the appendix for maximum block spacing.) The factors which determine the bending moment are discussed in the following paragraphs.

If the rib were composed of pin-connected curved segments as indicated in *Fig. 131,* bending moment would be zero at blocking points, and maximum $\mathbf{M}_t$ between, being equal to the product of the thrust **T** and rise **h** of the arc,

$$\mathbf{M}_t = \mathbf{Th} \qquad (1)$$

However, since the rib is continuous over several blocking points, there is a bending moment at each of these points acting counter to the bending moment between points. Equilibrium requires that the moment $\mathbf{M}_m$ at the midpoint between the blocking points be approximately equal to the difference between the moment $\mathbf{M}_t$ and the average $\mathbf{M}_b$ of the moments at the blocking points,

$$\mathbf{M}_m = \mathbf{M}_t - \mathbf{M}_b \qquad (2)$$

The bending moments contained in this equation can be computed by means of the theory of curved bars.[1] The distribution of the moments over the length of the arch is similar to that of the moments over a uniformly loaded straight beam, continuous over several supports. The moment $\mathbf{M}_m$ corresponds to the maximum bending moment between supports and $\mathbf{M}_b$ to the bending moments at the supports. The analysis led to the following conclusions regarding the maximum bending moment, $\mathbf{M}_{max}$, in semi-circular arch ribs bearing against equally spaced blocking points. If the rib is continuous and fixed at both ends, the maximum bending moment occurs at each blocking point. It is approximately equal to

$$\mathbf{M}_{max} = \mathbf{M}_b = 0.67\,\mathbf{M}_t = 0.67\,\mathbf{Th} \qquad (3)$$

wherein **T** is the thrust and **h** is the rise of the arc between blocking points.

If the rib is hinged at the crown or at the crown and both ends, the maximum bending moment occurs at the blocking points adjoining a hinge, such as blocking point 5 in *Fig. 134.* At each of these blocking points, the moment is roughly equal to

$$\mathbf{M}_{max} = \mathbf{M}_b = 0.86\,\mathbf{M}_t = 0.86\,\mathbf{Th} \qquad (4)$$

Equations 3 and 4 are also approximately valid, if the space between intermediate blocking points such as 2, 3, 4 and 5 in *Fig. 134* is less than the space between the hinge and the next blocking point. If this conditions prevails, **h** in *eqs. 3* and *4* denotes the rise of the arc in the hinged or end-sections.

Fig. 134 represents an arch rib which is hinged only at the crown point. In accordance with the preceding discussion, the maximum bending in such a rib occurs at the first blocking point away from the crown joint block provided the distance between blocking points either side of this point is the maximum permitted by the assumption. The point is marked No. 5 and according to equation 4 the bending moment at that point is equal to $0.86\,\mathbf{M}_t$.

1. See for instance S. Timoshenko, Strength of Materials, Part II. D. Van Nostrand Co. Ltd., 2nd edition, 1941,

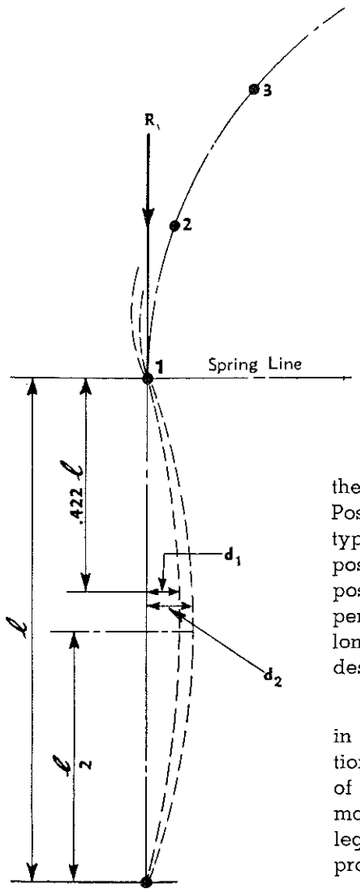


Fig. 133—Deflection of leg

Maximum fibre stress in the arch

The maximum fibre stress is equal to the sum of the compressive stress due to the thrust and the maximum compressive stress due to bending.

Stresses in straight leg (no side pressure acting)

The straight leg of a continuous rib represents a slender column acted upon by a vertical axial force equal to the thrust in the arch and by a bending moment due to continuity between leg and arch.

As a general rule the profile suitable for the arch will be satisfactory for the leg. This is because the stresses become critical only in extremely long legs. In such a case, dimensional limitations would prohibit shipping and the type of support would be changed to the Rib and Post, or the Rib, Wall Plate and Post types. In these types, there is no continuity between arch rib and post, and no bending moment is transmitted to the post. It acts as a long column and is designed independently of the arch rib applying the well known long column formulas commonly used in structural design.

An accurate computation of the critical stresses in the leg of a continuous rib would be out of proportion to the uncertainties associated with the estimate of the acting forces. For those interested, an approximate conception as to the degree of safety of the leg can be obtained by means of the following procedure.

First the allowable compressive stress f_{red} for the leg is computed on the assumption that the leg acts as a slender, concentrically loaded column. Then the maximum fibre stress f_c due to direct compression and to bending at midheight of the leg is computed. If f_c is smaller than f_{red} the leg may be considered safe.

The following considerations enter into the computation of stresses in the leg.

1. Because of continuity with the arch portion there is a bending moment $\mathbf{M_1}$ in the leg. This is maximum at the top, decreasing uniformly to zero at the foot. (It is assumed that the lowest arch blocking point is at the junction of the leg and arch.) This bending moment causes the leg to deflect toward the middle of the tunnel a distance $\mathbf{d_1}$ as shown in *Fig. 133*. The point of maximum deflection is .422 l below the top of the leg where l is the length of the leg.

The intensity of the moment $\mathbf{M_1}$ varies with the arrangement of the blocks on the arch. The maximum value, only, need be considered. $\mathbf{M_1}$ is the maximum when the first blocking point above the spring line is at the maximum permitted distance and the arch conditions are maximum as previously defined.

If the leg were perfectly rigid, the bending moment at the upper end of the leg would be about .67 $\mathbf{M_t}$. However, the leg is free to deflect as in *Fig. 133* and the effect of this deflection is to reduce the bending moment to less than 0.67 $\mathbf{M_t}$. Therefore the value $\mathbf{M_1} = 0.67\ \mathbf{M_t}$ is on the safe side.

2. The leg is loaded also by the vertical thrust $\mathbf{R_v}$ exerted by the arch at the spring line. This force not only causes axial compression, but, since the leg is now "bowed," increases the bending moment which in turn increases the deflection from $\mathbf{d_1}$ to $\mathbf{d_2}$, *Fig. 133*. The increase in the deflection from $\mathbf{d_1}$ to $\mathbf{d_2}$ is commonly so very small in the range of leg lengths and beam sizes commonly encountered that it increases the bending moment less than 3% and therefore can be neglected.

Since the maximum deflection occurs at a point between 0.422 l and 0.5 l below the top of the leg, the total bending moment at that point is equal to

$$\mathbf{M_c} = (1.0 - 0.422) \times 0.67\ \mathbf{M_t} + \mathbf{Td_2}$$

Since $\mathbf{M_t} = \mathbf{Th}$ *(equation 1)*,

$$\mathbf{M_c} = 0.38\ \mathbf{Th} + \mathbf{Td_2} \qquad (5)$$

At the upper end of the leg the moment is 0.67 **Th**. If the length of the leg does not exceed that of the leg of a customary continuous 2-piece continuous rib, the value $0.38\,\mathbf{h} + \mathbf{d_2}$ is always very much smaller than 0.67 **h**, or in other words, $\mathbf{M_c}$ is always smaller than the moment at the top of the leg, $\mathbf{M_1} = 0.67\ \mathbf{Th}$. Yet the rib is dimensioned such that it can even stand a bending moment $\mathbf{M_{max}} = 0.86\ \mathbf{Th}$ *(equation 4)*. Therefore, the maximum stress in the legs of a continuous rib is always smaller than the allowable stress.

For those interested in approximating the stresses in the leg, the computations are set forth under *Example No. 1* in the next chapter.

Side pressure on legs

Only occasionally is side pressure encountered in a rock tunnel. *Example No. 4* in *Chapter 11* shows the serious reduction of capacity of a straight legged rib when called upon to resist side pressure. Where slight side pressure is anticipated, the leg of the rib should be curved and blocked at suitable intervals. To prevent inward movement at the bottom, the foot may be pocketed or an invert strut provided. When the set is wedged tight, the legs exert an outward force against the rock to counteract side pressure.

If heavy side pressure is encountered, conversion to full circle ribs should be seriously considered.

CHAPTER 11

DETAIL PROCEDURE FOR AND EXAMPLES OF COMPUTATION OF STRESSES

INTRODUCTION

In this chapter a method of computing stresses in ribs will be explained step by step. To illustrate this explanation the stresses in a 2-piece continuous rib for a tunnel with a semi-circular roof, *Example No. 1*, will be computed. *Examples No. 2* and *No. 3* illustrate the method as applied to other shapes of tunnels using other types of support. All data referring to the first example will be preceded by the notation *Example No. 1*.

Example No. 4 is included to show the effect of side pressure on straight-legged ribs and the beneficial effect of converting the tunnel cross-section to a full circle when side pressures are encountered.

EXAMPLE NO. 1 24 FT. x 27 FT. TUNNEL

Construction of load diagram

Example No. 1 See *Fig. 134*, a tunnel 24 ft. wide, semi-circular arch, 13 ft. vertical side walls. Crown is assumed to be hinged and the ends of arch are fixed.

1. Draw the outside "design" line of the concrete for half the tunnel.

2. Draw in the rib, with its outside flange on the outside design concrete line. Assume 1" rib depth for each 3 feet of tunnel width. Draw neutral axis of rib.

Example No. 1 8" rib.

3. Draw in the "overbreak" line parallel to outside flange of rib.

Example No. 1 8" offset from rib.

4. Assume the maximum blocking point spacing and indicate the blocking points on the outside flange of the rib. One blocking point should be at the junction between arch and straight leg (at or near the spring line); another located not more than 6 in. from the crown joint. Label the blocking points 1, 2, 3, etc., starting at the spring line.

As the rib must be designed for maximum conditions, the second and third blocking points from the crown joint should be at the maximum spacing from the first blocking point. This induces maximum thrust in the arch. A maximum blocking space should separate the spring line blocking point from the one next above to induce maximum bending moment in the straight leg.

Example No. 1 50 in. maximum blocking point spacing, assumed.

5. Project blocking points radially to the overbreak line and neutral axis.

6. Assume the upper boundary of the body of rock which is to be carried by the rib. Draw vertical lines upward from the mid-points between the overbreak blocking points. The panels thus obtained represent the volume of rock acting at each blocking point.

Example No. 1 10 ft. of rock.

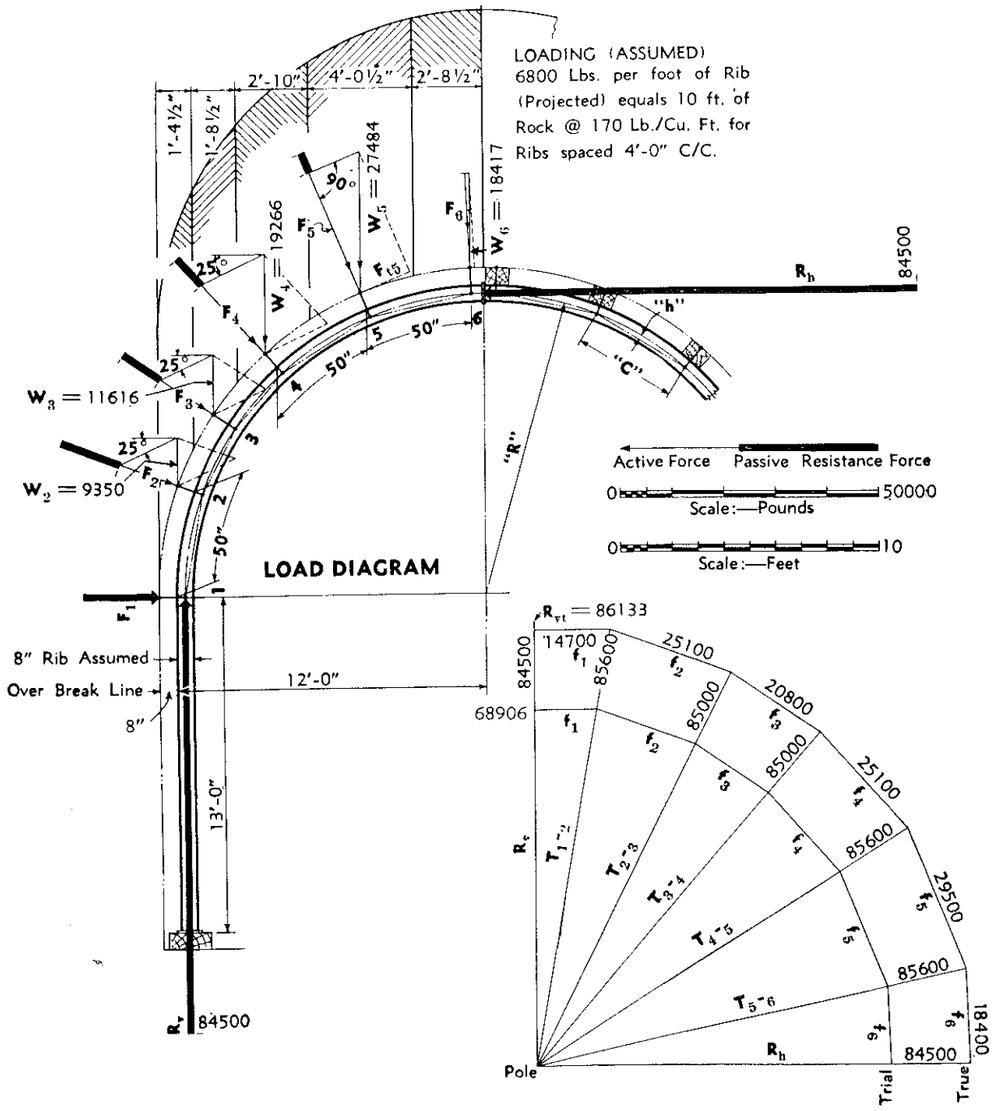


Fig. 134

7. Assume a rib spacing, and weight per cubic foot of rock. (Table IV at the end of this chapter gives weights of various rocks.) Compute the vertical load at each point and, to some suitable scale of pounds per inch, show each load acting vertically on the overbreak blocking point, as $\mathbf{W}_5$ in *Fig. 134.*

Example No. 1 10 ft. of rock; 4 ft. rib spacing; weight of , 170 lbs. per cu. ft. = 6800 lb. per ft. of rib, projected on a horizontal line.

8. As shown in *Fig. 131* resolve each vertical force **W** at the overbreak blocking point into a radial load force **F** and a tangential component $\mathbf{F}_t$ if the tangent is inclined less than 25° to the horizontal, otherwise a 25° component as in *Fig. 130 h.*

9. Show an upward vertical reaction $\mathbf{R}_v$ at the spring line *(point 1).*

10. Draw in the chords connecting the neutral axis blocking points. These chords represent the direction of the thrust in each panel.

Construction of force polygon

11. From a common point (pole) draw a vertical ray $\mathbf{R}_v$ and rays parallel to each thrust line (chord) and label them $\mathbf{T}_{1\text{-}2}$, $\mathbf{T}_{2\text{-}3}$,etc. End up with a horizontal ray $\mathbf{R}_h$.

12. Compute the sum $\mathbf{R}_{vt}$ of all the vertical loads **W** which act on one-half of the tunnel roof. According to *Fig. 134* the roof carries only one of the two components of each of the weights **W**. Therefore the total vertical pressure $\mathbf{R}_v$ on one-half of the roof is smaller than $\mathbf{R}_{vt}$. However, its real value is not yet known. Hence, it is necessary to construct a trial polygon starting at an arbitrarily selected point on the ray $\mathbf{R}_v$. For the sake of convenience, it is advisable to assume this point at a distance 0.80 $\mathbf{R}_{vt}$ from the pole. Starting at this point draw a line $\mathbf{f}_1$ (parallel to force $\mathbf{F}_1$ on the load diagram) to intersect ray 1-2. From this intersection draw a line $\mathbf{f}_2$ parallel to force $\mathbf{F}_2$ to intersect ray 2-3, and repeat until the horizontal ray $\mathbf{R}_h$ is reached, thus completing the trial force polygon.

Example No. 1 $\mathbf{R}_{vt}$ is 86133 lbs. 80% of $\mathbf{R}_{vt}$ is 68906 lbs.

13. Now compare $\mathbf{f}_2$ on the polygon with force $\mathbf{F}_2$ on the load diagram, $\mathbf{f}_3$ with force $\mathbf{F}_3$ and so on. Usually at one or more points the radial component **F** on the load diagram will exceed the corresponding trial polygon force **f**.

14. Transfer all forces **F** which exceed the corresponding trial polygon forces **f** to the polygon, extending the rays as necessary, and complete a new polygon for the force **f** farthest removed from the pole. This is the *true force polygon.* It will be noted that all the other true polygon forces **f** are now greater than the corresponding forces **F** on the load diagram. The excess represents the force of passive resistance mobilized by the rock against the active tendency of the rib to advance toward the rock.

Example No. 1 Maximum excess is at point 6. In *Fig. 134* the passive resistance furnished by the rock has been plotted as the thick part of the arrow.

Determination of thrusts

15. The length of the rays of the true force polygon represent thrust. Since the design must be based on the most unfavorable stress conditions, determine which is the longest ray.

Example No. 1 Thrust is uniform and maximum in all panels of maximum block spacing because of constant curvature of rib. Thrust scales 85600 lbs.

Note: The excess of R_{vt} over R_v in *Fig. 134* represents that part of the rockload on the roof which is transferred by friction forces onto the rock located next to the tunnel. It is always very small compared to the total load which the friction forces could carry. Thus, for instance, in *Fig. 134* the excess amounts to about 0.02 R_h whereas the friction forces could carry at least $R_h \times \tan 25^\circ = 0.45\, R_h$. This is due to the fact that the vertical component of the passive forces (thick part of the arrows F_2 to F_5) relieves the load which would otherwise be carried by the friction forces.

Bending moment

16. As stated in *Chapter 10* it is not worth while to determine the intensity of the various moments as the rib, being of constant section, must be proportioned for the maximum. This maximum occurs at the blocking point next to the crown joint blocking point. The bending moment M_{max} cannot exceed .86 M_t and for design purposes may be taken as .86 M_t.

Example No. 1 Maximum moment is at point 5.

Maximum total stress

17. The sum of the compressive stress due to the thrust and the maximum compressive stress due to bending is the maximum total stress.

Computations for stresses in arch rib

18. The following symbols are used in the formulas below.

C = Chord length between neutral axis blocking points, in inches, (measured on the load diagram or computed).

Example No. 1 48.4 in., computed.

R = Radius of neutral axis of rib, in inches, from load diagram.

Example No. 1 140.0 in.

h = Rise of arc between blocking points, in inches, computed.

M_t = Bending moment in inch-pounds if rib sections could be pin connected at blocking points, computed.

M_{max} = Maximum bending moment, in inch-pounds, in rib continuous for at least 4 blocking points, computed.

T = Thrust, in pounds, scaled from true force polygon.

Example No. 1 85600 lbs.

S = Section Modulus of beam under consideration, from structural hand book.

Example No. 1 8 x 5¼ in. WF-beam, 21 lbs. per foot.
$S = 18.0$ in.^{3}

A = Sectional Area of beam under consideration, less holes, in sq. inches, from structural hand book, reduced by computation.

Example No. 1 $A = 6.18$ sq. in. less .22 sq. in. for 7/8 in. tie rod hole $= 5.96$ sq. in.

f_r = Stress in arch portion of rib, in pounds per sq. in., computed.

The following formulas are used in stress computations:

$$h = R - \sqrt{R^2 - \left(\frac{C}{2}\right)^2}$$

Example No. 1 $h = 140 - \sqrt{140^2 - \left(\frac{48.4}{2}\right)^2} = 2.1$ in.

$M_t = hT$ *(equation 1)*

Example No. 1 $M_t = 2.1 \times 85600 = 179{,}760$ in.-lbs.

$M_{max} = .86\, M_t$ *(equation 4)*

Example No. 1 $M_{max} = .86 \times 179760 = 154594$ in.-lbs.

$$f_r = \frac{T}{A} + \frac{M_{max}}{S}$$

Example No. 1 $f_r = \frac{85600}{5.96} + \frac{154594}{18} = 22950$ lbs. per sq. in.

f_{all} = Allowable fibre stress = 24000 p.s.i., for temporary construction. (May be set at any desired figure.)

Example No. 1 Since the stress f_r of 22950 p.s.i. is less than the allowable stress f_{all} of 24000 p.s.i., this rib is considered satisfactory.

Leg checked as a column

19. Having thus selected the beam to be used for the arch, it may be interesting to investigate whether this beam is capable of acting as a column to carry the forces down to subgrade. In *Chapter 10* it is shown that the leg is normally amply strong, but a check is made here for the benefit of those interested.

First, the permitted stress in the leg is computed by means of one of the customary column formulas. This computation takes care of lack of true homogeneity in the metal, variation in the shape and thickness of the section, and eccentricity of application of the forces. All of these departures from perfect conditions cause the axial force to set up a bending moment in the leg. This moment is maximum at the middle of the leg and requires reduction of the allowable stress.

Second, the stress in the leg is computed and compared to the reduced allowable stress.

20. The following symbols are in addition to those in *Paragraph 18.*

f_{red} = Maximum permitted fibre stress at mid-point, in lbs. per. sq. in., computed.

f_{all} = Allowable fibre stress = 24,000 p.s.i., for temporary structures, assumed.

M_1 = Bending moment at spring line in inch-lbs., computed.

M_c = Bending moment at point of maximum deflection of leg in upper half of middle third, in inch-lbs., computed.

d_1 = Deflection of the leg caused by bending moment M_1, in inches, computed.

d_2 = Total deflection of leg in the upper half of the middle third, caused by the sum of the moments, in inches, computed.

E = Modulus of Elasticity of the structural steel used in the rib, which is 29,400,000 lbs. per sq. in.

I = Moment of Inertia of the beam under consideration, from structural hand book.

Example No. 1 $I = 73.8$ in.^{4}

R_v = Thrust acting on the leg, in pounds, scaled from true force polygon.

Example No. 1 $R_v = 84500$ lbs.

l = Length of leg below spring line blocking point, in inches, from load diagram.

Example No. 1 $l = 156$ in.

f_1 = Stress in leg at spring line, in lbs. per sq. in., computed.

f_c = Stress in the leg at point of maximum deflection, in lbs. per sq. in., computed.

r = Radius of gyration for the major axis of the section, from structural hand book.

Example No. 1 $r = 3.45$ in.

Stresses in leg

21. *Reduction of allowable stress.* Two commonly used formulas for reduction of allowable stress in columns are given below.

$$f_{red} = \frac{f_{all}}{1 + \frac{l^2}{f_{all}\, r^2}}$$

Example No. 1 $$f_{red} = \frac{24000}{1 + \frac{156^2}{24000 \times 3.45^2}} = 22{,}115 \text{ p.s.i.}$$

This is the American Institute of Steel Construction formula for Strut Loading.

$$f_{red} = \frac{f_{all}}{18000} \times \left[17000 - .485 \left(\frac{l}{r}\right)^2\right]$$

Example No. 1

$$f_{red} = \frac{24000}{18000} \times \left[17000 - .485 \left(\frac{156}{3.45}\right)^2\right] = 21{,}345 \text{ p.s.i.}$$

This is the well-known straight line formula adjusted to an allowable fibre stress other than 18000 p.s.i. This straight line formula will ordinarily give somewhat lower values than the A.I.S.C. formula in the range of l/r commonly encountered in rib design.

22. *Computation of stresses in leg.* The following formulas are used.

$M_1 = .67\, M_t = .67\, hT$

Example No. 1 $M_1 = .67 \times 2.1 \times 85600 = 120{,}440$ in.-lbs.

$$f_1 = \frac{T}{A} + \frac{M_1}{S}$$

Example No. 1 $$f_1 = \frac{85600}{5.96} + \frac{120440}{18.0} = 21050 \text{ p.s.i.}$$

$$d_1 = .0642\, \frac{M_1\, l^2}{E\, I}$$

Example No. 1 $$d_1 = .0642 \times \frac{120440 \times 24{,}336}{29400000 \times 73.8} = .087 \text{ in.}$$

$$M_c = \frac{(.578\, M_1)^2}{.578\, M_1 - d_1\, R_v}$$

Example No. 1 $$M_c = \frac{(.578 \times 120440)^2}{.578 \times 120440 - .087 \times 84500} = 77800 \text{ in.-lbs.}$$

$$f_c = \frac{R_v}{A} + \frac{M_c}{S}$$

Example No. 1 $$f_c = \frac{84500}{5.96} + \frac{77800}{18.0} = 18500 \text{ p.s.i.}$$

To be satisfactory, the stress in the leg must satisfy both conditions below.

f_1 must be less than f_{all}

Example No. 1 21050 is less than 24000

f_c must be less than f_{red}

Example No. 1 18500 is less than 21345 (or 22115)

Thus the beam selected for the arch rib is also quite satisfactory for the leg.

EXAMPLE NO. 2 DOUBLE TRACK RAILROAD TUNNEL

For this example the standard cross-section for a double track railroad tunnel as shown in the A. R. E. A. handbook is chosen. It differs from *Example No. 1* in that the arch is not semi-circular, and that due to its size, the rib and post type of support is used. *Fig. 135* illustrates this example and sets forth the various assumptions as to loading, blocking point spacing, rib size, etc. The same symbols are used as in the previous example.

C = Chord of 3′-7″ blocking point space at haunch = $41\frac{3}{8}$ in., computed.

R = Radius to neutral axis of rib at haunch = 9′-10″ = 118 in.

S = Section Modulus of rib. For 8 x 8 in. WF-beam 40 lbs. per foot, $S = 35.5$ in.^{3}

A = Sectional area of rib, reduced for tie rod holes. $A = 11.44$ sq. in.

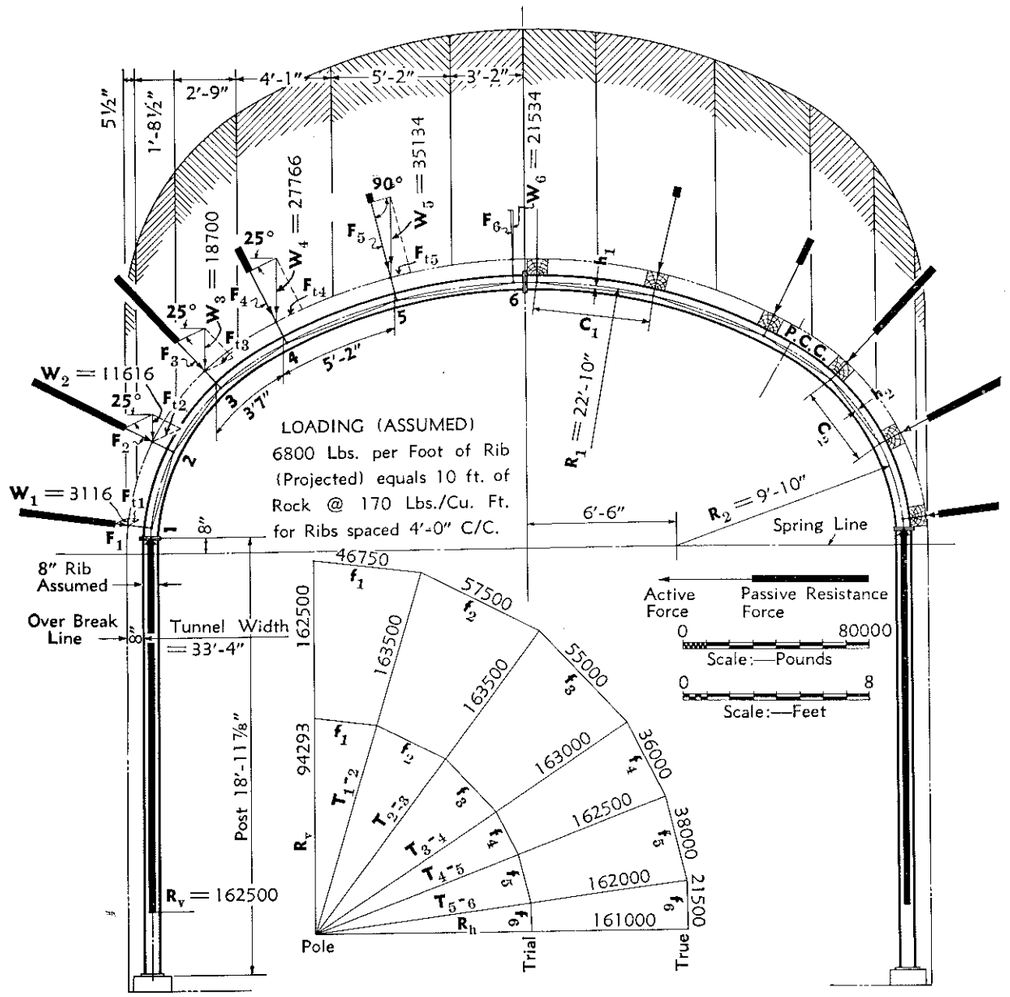


Fig. 135

f_{all} = Allowable fibre stress in rib = 24000 p.s.i.

Maximum conditions prevail at point No. 2.

T = Thrust = 163500 lbs., scaled from force polygon.

Computations for maximum stress in the arch

$$h = 118 - \sqrt{118^2 - \left(\frac{41.375}{2}\right)^2} = 1.8 \text{ in.}$$

$M_t = 1.8 \times 1635000 = 294300$ in.-lbs.

$M_{max} = .86 \times 294300 = 253098$ in.-lbs.

$$f_r = \frac{163500}{11.44} + \frac{253098}{35.5} = 21422 \text{ p.s.i.}$$ which is considered satisfactory.

Computations for stress in the post.

Assume an 8 x 8 in. WF-beam at 31 lbs. per foot for the post.

A = 8.87 sq. in.

r = 3.47 in.

l = Length of post = 227 in.

R_v = 162500 lbs., scaled from true force polygon.

$$f_c = \frac{162500}{8.87} = 18320 \text{ p.s.i.}$$

$$f_{red} = \frac{24000}{18000} \times \left[17000 - .485\left(\frac{227}{3.47}\right)^2\right] = 19898 \text{ p.s.i.}$$

Since f_c is less than f_{red}, this post is considered satisfactory.

EXAMPLE NO. 3 TWO LANE HIGHWAY TUNNEL

For this example, the cross-section of the Pennsylvania Turnpike Tunnels is used. The arch departs from the conventional semi-circular shape to provide for a ventilating duct. The 10 foot rock load assumed herein may be greater than the loads actually encountered in the driving of these tunnels. It is used, however, for the sake of comparison with other examples. *Fig. 136* illustrates this example.

C = Chord of 5'-9" blocking point space = 66.84 in., computed.

R = Radius to neutral axis of rib = 187 in.

S = Section Modulus of rib. For 10 x 8 in. WF-beam 41 lbs. per foot, S = 44.5 in.^{3}

A = Sectional area of rib reduced for tie rod holes. A = 11.78 sq. in.

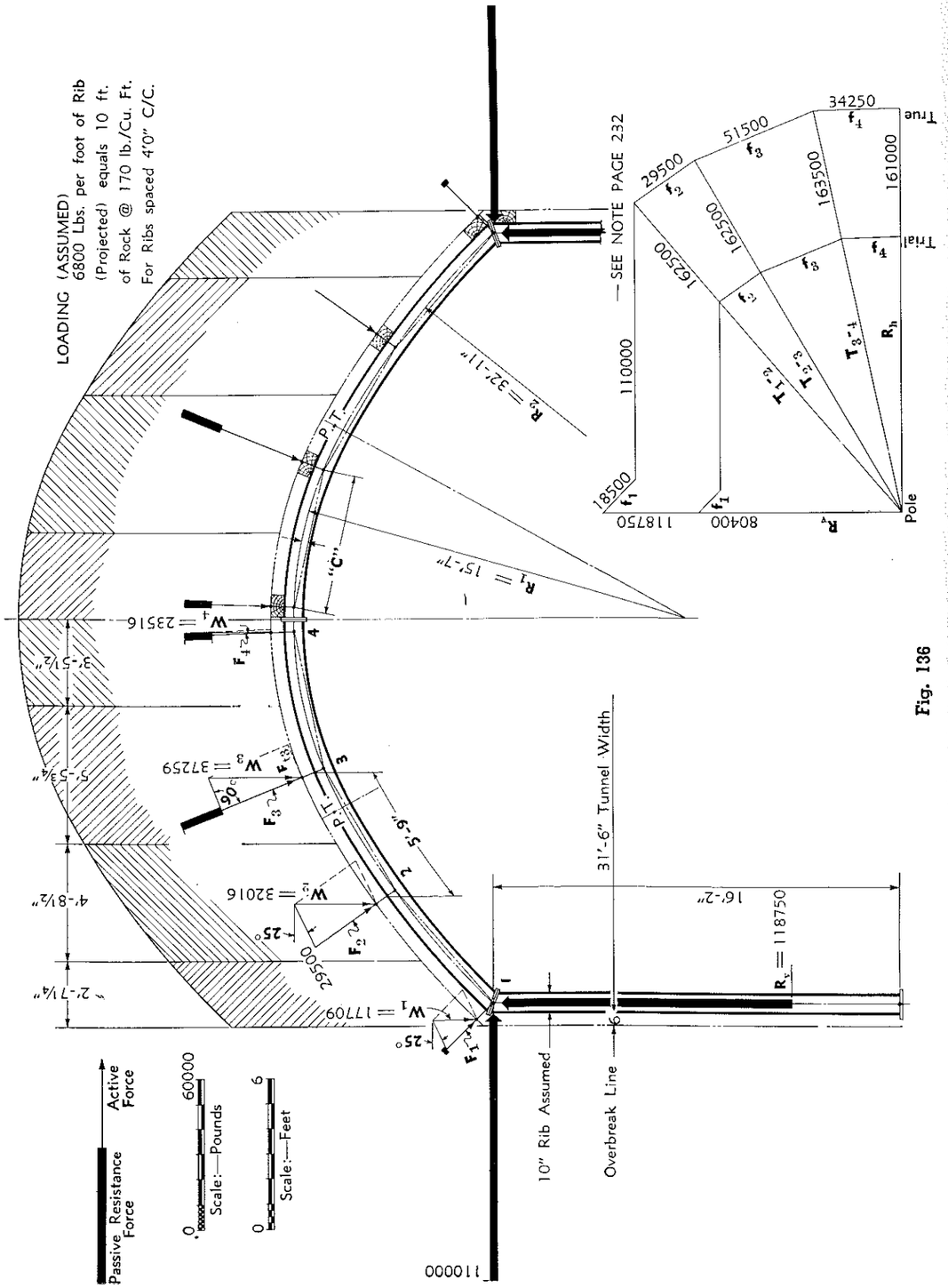


Fig. 136

f_{all} = Allowable fibre stress in rib = 24000 p.s.i.

Maximum conditions prevail at point 3.

T = Thrust = 163500 lbs., scaled from true force polygon.

Computations for maximum stress in the arch

$$h = 187 - \sqrt{187^2 - \left(\frac{66.84}{2}\right)^2} = 3.1 \text{ in.}$$

$M_t = 3.1 \times 163500 = 506850$ in.-lbs.

$M_{max} = .86 \times 506850 = 435891$ in.-lbs.

$$f_r = \frac{163500}{11.78} + \frac{435891}{44.5} = 23674 \text{ p.s.i.,}$$ which is considered satisfactory.

Computations for stress in the post

Assume 10 x 5¾ in. WF-beam at 21 lbs. per foot.

A = 5.97 sq. in.

r = 4.14 in.

l = length of post = 194 in.

R_v = 118750 lbs., scaled from true force polygon.

$$f_c = \frac{118750}{5.97} = 19891 \text{ p.s.i.}$$

$$f_{red} = \frac{24000}{18000} \times \left[17000 - .485\left(\frac{194}{4.14}\right)^2\right] = 21300 \text{ p.s.i.}$$

Since f_c is less than f_{red}, this post is considered satisfactory.

EXAMPLE NO. 4—TUNNEL SUPPORT SUBJECT TO SIDE PRESSURE

This example sets forth the computations made in connection with *Fig. 115* page 186 to illustrate the advantages of a full circle rib over a straight legged rib when side pressures are encountered. Where side pressure is present it is necessary to lag tight and pack solid, as would be the case in squeezing rock. Hence the blocking point spacing is reduced to zero and because of this, there are no bending stresses in the curved portion of the rib.

In the following paragraphs, the maximum stress in the rib is computed for each type under vertical and under combined loading. This is done for 10 ft. of rock, weighing 170 lbs. per cu. ft. with ribs spaced at 4 ft. The respective carrying capacities in ft. of rock is then obtained by increasing the load till the fiber stress reaches the permitted stress.

Case 1 Vertical load only on straight legged rib (shown on left side of *Fig. 115.*)

An inspection of the force polygon shown in *Example 1*, where a 50-in. blocking point spacing was used, reveals that the thrust **T** is less than the total vertical reaction R_{vt}. As the blocking point spacing is reduced the thrust **T** increases until at zero spacing $T = R_{vt}$.

R_{vt} = the vertical load on half the tunnel.

$$T = R_{vt} = \frac{24}{2} \times 4 \times 10 \times 170 = 81600 \text{ lbs.}$$

$A = 8.870$ sq. in., for 8 x 8 in. WF-beam, 31 lbs. (after deducting for tie rod hole)

$$f_r = \frac{T}{A} = \frac{81600}{8.87} = 9200 \text{ p.s.i.}$$

Inasmuch as there is no bending moment in the arch rib, f_r is also the stress in the straight leg, or $f_r = f_c$. However, the leg is considered to be a slender column and whereas the allowable fibre stress f_{all} in the arch may be taken as 24000 p.s.i. the fibre stress f_{red} permitted for the leg is smaller on account of the danger of buckling.

$r = 3.47$ in., for 8 x 8 in. WF-beam, 31 lbs.

$l = 114.8$ in., the length of the straight leg.

$$f_{red} = \frac{24000}{18000}\left[17000 - .485\left(\frac{l}{r}\right)^2\right] = 21900 \text{ p.s.i.}$$

As f_c cannot exceed f_{red}, the leg limits the capacity of the rib set. The capacity H_p in feet of rock then is

$$H_p = \frac{f_{red}}{f_c} \times 10 \text{ feet of rock or } \frac{21900}{9200} \times 10 = 23.8 \text{ ft.}$$

Case 2 Side unit pressure equal to 1/3 the unit vertical pressure acting on the straight legged rib (shown at the left in *Fig. 115.*)

The thrust **T** in the arch remains the same as in *Case 1* as the vertical load on the arch rib causes the rib to press outward against the rock with a greater force than the assumed inward horizontal pressure. (Note: This statement is true for the ratios of leg length to arch rise and horizontal pressure to vertical pressure in this example. A longer leg and/or greater ratio of horizontal to vertical pressure may vitiate this statement.)

There is no bending moment transmitted into the leg from the arch as in *Example 1*. Bending and deflection in the leg does result, however, from the horizontal pressure. Hence the leg is subjected to combined stress, i.e., compression due to the thrust **T** transmitted into it from the arch rib, and bending stress from two causes, one being from the active external horizontal ground pressure, and the other from the thrust acting about the deflection resulting from the first.

Since the thrust **T** and consequently the stress in the arch is known, as explained in *Case 1*, the following computations deal only with the stresses in the leg, which limit the capacity of the rib set. For the sake of simplicity the straight leg is considered to be vertical.

The following additional symbols and formulas are used in the computations.

M_h = Bending moment caused by lateral pressure, in in.-lbs. M_h is maximum at .125 x l below mid-point of leg.

M_c = Bending moment from all causes combined at point of maximum deflection, in in.-lbs. The maximum deflection is slightly below mid-point of leg.

d_1 = Maximum deflection of leg caused by moment M_h in inches. This occurs at .078 x l below mid-point of leg.

d_2 = Total maximum deflection, from all causes combined, in inches. Maximum slightly below mid-point of leg.

E = Modulus of Elasticity, 29,400,000 p.s.i.

I = Moment of Inertia, 109.7 in.^{4}, for 8 x 8-in.WF-beam, 31 lbs.

$R_v = R_{vt} = T$ = Thrust acting on leg, 81600 lbs.

f_c = Stress in leg at point of maximum deflection, in p.s.i.

W = Total horizontal pressure of rock on straight leg, in pounds.

H_p = Height of rock causing vertical load, 10 ft.

w = Weight per cu. ft. of rock, 170 lbs.

l = Length of straight leg in inches, = 114.8 in.

s = Rib spacing, 4 ft.

$$W = \frac{sH_pwl}{3 \times 12} = \frac{4 \times 10 \times 170 \times 114.8}{3 \times 12} = 21700 \text{ lbs.}$$

$M_h = \frac{Wl}{14.2}$ for a beam fixed at one end and with simple support at the other $= \frac{21700 \times 114.8}{14.2} = 175{,}440$ in.-lbs. This occurs at .125 x l or 14.3 in. below mid-point of leg.

$$d_1 = \frac{Wl^3}{185EI} = \frac{21700 \times 1512954}{185 \times 29400000 \times 109.7} = .055 \text{ in. This occurs at .078 x l or 9 in. below mid-point of leg.}$$

The above moment and deflection are increased by the thrust **T** acting around d_1 as a lever arm. These are both maximum near the mid-point of the leg so that when added to M_h the resulting total moment M_c is maximum just below the mid-point. It is not important to find out just where.

$$M_c = \frac{M_h^2}{M_h - d_1R_{vt}} = \frac{175440^2}{175440 - .055 \times 81600} = 180{,}050 \text{ in. lbs.}$$

$$f_c = \frac{R_{vt}}{A} + \frac{M_c}{s} = \frac{81600}{8.87} + \frac{180050}{27.4} = 9200 + 6571 = 15770 \text{ p.s.i.}$$

This stress was caused by 10 ft. of rock. The capacity of the rib set then is

$$H_p = 10 \times \frac{f_{red}}{f_c} = \frac{10 \times 21900}{15770} = 13.9 \text{ ft.}$$

Thus it may be seen that when side pressure is acting the straight leg reduces considerably the load which may be carried by a horseshoe shaped rib as shown in *Fig. 115.*

This limitation may be removed by converting to a circular extrados as shown at the right of *Fig. 115.* Design for the circular shape is described below.

Case 3 Full circle rib

A circular rib, using the same rib profile as the horseshoe shape is drawn over the horseshoe layout, allowing for minimum concrete thickness at the bottom corners and the crown. The size required for the 24 ft. by 24 ft. tunnel under consideration is 26 ft. O. D.

Since the ribs are fully lagged and dry packed there is no bending moment in the rib. Passive resistance of the surrounding ground causes the thrust line to pass through the centroid of the rib section at all points as previously described. Therefore, the stress is entirely compressive, and is equal to one-half the vertical load. (Horizontal thrust in the ring due to vertical load is greater than the active horizontal load from the ground.)

The number of feet of rock which the rib will carry is determined from the compression strength of the rib.

$$T = R_{vt} = Af_{all} = 8.87 \times 24000 = 212{,}880 \text{ lbs.}$$

$$H_p = \frac{2T}{(O.D.)\ sw} = \frac{2 \times 212880}{26 \times 4 \times 170} = 24.1 \text{ feet of rock}$$

Note A (Fig. 136) Page 228

The sloping portion of force f_1 in the force polygon is obtained by drawing a line 25 degrees from horizontal, starting at the top of vertical load line W_1 on the load diagram until it intersects line F_1. The magnitude thus obtained is the active resultant of load W_1 applied in a direction normal to the tunnel support at load point 1. This force is transferred to the force polygon.

A horizontal line drawn from the lower end of sloping line f_1 on the force polygon intersects ray $T_{1\text{-}2}$. The relationship of the two forces comprising f_1 will always be constant, the total value including active and passive components depending upon the critical load which establishes equilibrium of the arch.

TABLE IV

Weights of Rocks in pounds per cubic foot

Coarse-grained igneous rocks		Fine grained igneous rocks	
Diorite	170 to 186	Andesite	159 to 178
Gabbro	178 to 195	Basalt	175 to 192
Granite	145 to 176	Rhyolite	131 to 169
Syenite	164 to 181	Trachyte	160
Metamorphic rocks		Sedimentary rocks	
Gneiss	165 to 182	Conglomerate	137 to 150
Marble	162 to 175	Dolomite	131 to 168
Quartzite	162 to 168	Limestone	131 to 168
Schist	168 to 182	Sandstone	125 to 168
Slate	162 to 175	Shale	125 to 168
		Crushed rock, saturated	125 to 150

SECTION IV

APPENDIX

CATALOGUE

of

STEEL TUNNEL SUPPORT

for

ROCK TUNNELS

as

MANUFACTURED

by

COMMERCIAL SHEARING, INC.

of

YOUNGSTOWN

OHIO

IMPORTANCE OF STRUCTURAL DETAILS

To attain the most economical results it is not enough to select the most efficient types of steel support. The details of that support are equally important. Safety requires that it be possible to erect a set in *minutes*. Whatever meets this fundamental condition automatically produces low erection cost and speedy tunneling.

Simplicity of design and detail essential

For speed of erection in the tunnel it is essential to

(1) design the support system with the minimum number of individual members, hence the minimum number of joints;

(2) design the joints with the utmost simplicity and the absolute minimum number of bolts;

(3) fabricate the members with ample bolt and wrench clearances and easy fits. Time-consuming close fits are not tolerable under the trying conditions inherent in tunnel support erection.

Bolts

Another detail which requires and receives special attention is the bolt. Bolts are used very sparingly, and generally as an aid to erection rather than a stressed part. Holes have generous clearance, much more than standard in structural practice. The threads are free-running and quick acting similar to those used for fitting up purposes. They will stand considerable abuse and the nut cannot be started cross-threaded. Heads are the same dimension across the flats as the nuts, thus the same wrench works on both.

Design and details of Commercial Support based on wide experience

The details of the steel supports illustrated on the following pages have been developed and selected from an experience gleaned from contact with several hundred tunnels over a period of nearly 50 years. Speed and ease of erection have been the prime considerations in mind in adopting each detail. Low fabricating and erection costs are the natural corollaries.

Many joints and attachments depart radically from conventional structural design practices. Each one, however, satisfies tunnel design requirements. The butt joint has practically displaced the spliced joint, and riveting has given way to welding and bolting. The simplicity resulting from this departure from tradition to functional design satisfies the peculiar requirements of tunnel support.

Commercial Standards

The details shown on the following pages are considered standard by Commercial having been developed and proven on many tunnel jobs. They are presented as a guide to Designers, Contractors and others who may be preparing plans, specifications or estimates for a tunnel project.

It is recognized, however, that special cases will require special treatment, so these details are not to be considered as "hard and fast" rules. Details are affected by the method of attack and by ground conditions, handling and erecting equipment, etc., so that a departure from the standard details may be advantageous. Commercial is equipped and willing to furnish supports whose details differ from those shown herein.

FABRICATING INFORMATION

All Ribs Cold Pressed

Commercial bends all ribs by pressing cold, in dies. This requires extremely heavy pressure, but produces better, stronger ribs.

Cold working steel greatly increases its elastic range, and practically eliminates the yield point. This means that the curved portion of the rib is stronger than it was before, and will carry much more load than a rib not cold worked.

Range of Bending Capacity

Commercial has tools for bending beams of any size up to 16" WF-beams and some tools for deeper sections; also press capacity for bending larger sections. Beams 30" deep have been bent but 27" is considered the practicable maximum depth because of dimensional limitations on present equipment.

Bending Limits

Minimum practical outside radii for cold bending of beams is as follows:

Sections	STEEL	
	Normal Grade ASTM Spec. A36-62 (Minimum Yield Point 36000 P.S.I.) Minimum Outside Radius	ASTM Spec. A7-61 (Minimum Yield Point 33000 P.S.I.) Minimum Outside Radius
I-Beams	12 x Rib Depth	9 x Rib Depth
H-Beams and WF-Beams	15 x Rib Depth	11 x Rib Depth

A rib may be bent to one or more radii with or without tangents at one or both ends.

Tolerances

The standard contour tolerance provides that for steel WF sections up to 14 in. deep weighing up to 150 lbs. per foot, rib segments shall conform to a true template at the ends but intermediate points may depart up to 3/8" from the true template, provided that no point shall depart more than 1/8" from a template section 3 feet long.

For heavier and larger sections, up to 27 in. deep departure of intermediate portions of rib segments from a true template shall not exceed 5/8" and deviation from a 3 foot long template shall not exceed 3/16 in.

RIB CAPACITIES

Rib Capacity Tables

Tables 1 and *2* on the following pages will be helpful in making a preliminary selection of rib size. They show the allowable vertical load, in pounds per lineal foot of rib, projected on the horizontal, when applied to the rib at blocking points spaced as indicated. The assumptions which were made when computing the table values are graphically represented in *Figs. 2* and *3* on page 239.

Data given is for tunnels with a semi-circular roof. Ribs for tunnels of other shapes should be investigated by the method described in *Chapter 11*, pages 219 to 232.

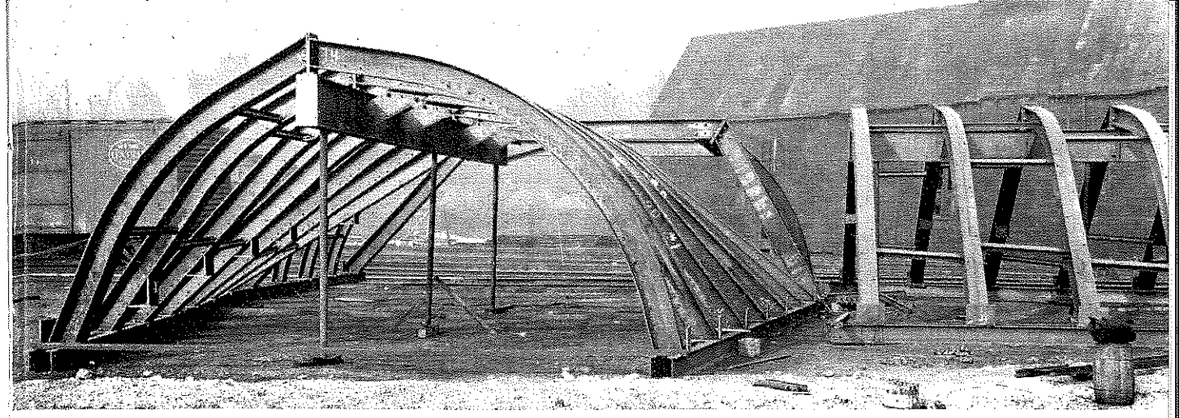


Fig. 1

Delaware Aqueduct—Board of Water Supply, New York City

KENSICO-WEST BRANCH TUNNEL

Contractor: Frazier-Davis Construction Co., *subcontractor* to S. A. Healy Co.

Support structure for enlargement at bottom of Shafts 15 and 16 partially erected prior to shipment. Section shown at the right belongs at the far end of the main structure in the foreground. Fig. 62 on page 126 shows this structure erected in the tunnel.

Span to outside of wall plates for normal tunnel, 19'-10". Span as enlarged at the shaft, 32 ft., to provide room for skip, by-pass, and rotary car dumper.

Rectangular frame in the background was provided for attachment of hoist framing.

Curved ribs in foreground and at right are 10" x 8" WF-beams, 45 lbs. Rectangular shaft frame, 12" x 10" WF-beams, 64 lbs. Curved ribs at shaft 8" x 8" WF-beams, 40 lbs. Wall plates, double 8" I-beams, 18.4 lbs.

The longitudinal I-beam and the angle brackets on the inside of the ribs were provided for temporary posts to support the structure till heavy posts could be placed under the wall plates in the enlargement.

Vertical blocking plates at the rib feet were convenient for placing horizontal thrust blocks.

The following points should be kept in mind when using the tables.

1. In all tables values are based on a maximum fiber stress of 27000 p.s.i.
2. Deductions have been made for tie rod holes in the web. If additional holes in the web or flanges are desired, the allowable loads should be reduced accordingly.
3. Blocking point spacings cannot be exceeded without reducing the rib capacity. Capacity will decline in proportion to the *square* of the blocking point spacing as it is increased; hence, the capacity falls off sharply if the spacing shown is exceeded. Conversely, closer spacing resulting from additional blocking points will increase the capacity rapidly.
4. No side loads are considered to be acting on the vertical legs.
5. Straight legs of the *Continuous Ribs* listed in *Table 1* are strong enough to carry the arch load indicated within the lengths that can be shipped by rail. Note: if the blocking point spacing is reduced to zero by packing, the capacity of the arch rib increases to a point where the leg becomes the limiting factor but at considerable higher values than shown in the table. (See *Example 4, Case 1*).
6. It is assumed that all wedges and blocks will be kept tight.

TABLE 1—CONTINUOUS RIBS

CAPACITY IN POUNDS PER FOOT OF TUNNEL WIDTH — MAXIMUM FIBER STRESS—27000 LBS. PER SQ. IN.

BEAM		Width of Tunnel to Outside Design Concrete Line															
Nominal Depth, Flange Width & Type	Wt. Per Foot	14′-0″	16′-0″	18′-0″	20′-0″	22′-0″	24′-0″	26′-0″	28′-0″	30′-0″	32′-0″	34′-0″	36′-0″	38′-0″	40′-0″	42′-0″	44′-0″
		40″	42″	44″	46″	48″	50″	52″	54″	56″	58″	60″	62″	64″	66″	68″	70″
		Maximum Blocking Point Spacing															
4″ I	7.7	3090	2780														
4″ x 4″ H	13.0	5380	4850	4400	4020												
5″ I	10.0	4530	4070	3690	3360												
5″ x 5″ Stanchion	16.0	7780	7000	6330	5770					Values shown are for stress of 27 Kips							
5″ x 5″ H	18.9	8840	7940	7190	6550					For 24 Kips multiply by .889							
6″ I	12.5	6290	5660	5110	4650	4260											
6″ I	17.25	7990	7180	6490	5920	5430	5010										
6″ x 4″ Light Beam	12.0	6200	5560	5020	4570	4200											
6″ x 4″ " "	16.0	8480	7600	6870	6270	5740	5300										
6″ x 6″ Stanchion	15.5	8380	7500	6780	6180	5660	5230										
6″ x 6″ H	20.0	10740	9630	8710	7930	7270	6700	6220	5780								
6″ x 6″ H	25.0	13270	11890	10770	9800	8980	8280	7680	7140	6670							
7″ I	15.3			6740	6130	5610	5190										
8″ I	18.4			8590	7820	7170	6610										
8″ I	23.0			10240	9330	8550	7890	7310	6790	6340	5940	5580	5250				
8″ x 4″ Light Beam	15.0			7110	6470	5930	5470										
8″ x 8″ H	34.3			16820	15310	14020	12940	12000	11160	10410	9750	9170	8640	8180	7740	7350	6990
8″ x 5¼″ W.F.	17.0			8220	7510	6880	6360	5890									
8″ x 5¼″ " "	20.0			9820	8940	8160	7550	7010	6500	6070							
8″ x 6½″ " "	24.0			11920	10860	9930	9170	8500	7900	7380	6920	6490	6120	5790	5490		
8″ x 6½″ " "	28.0			14010	12670	11600	10710	9930	9220	8610	8070	7580	7150	6770	6410		
8″ x 8″ " "	31.0			15550	14160	12970	11970	11090	10300	9630	9020	8470	7990	7560	7150	6790	6470
8″ x 8″ " "	35.0			17590	16030	14750	13540	12550	11670	10900	10200	9600	9040	8550	8100	7680	7310
8″ x 8″ " "	40.0			20100	18300	16750	15470	14330	13320	12430	11650	10960	10330	9760	9250	8770	8350
8″ x 8″ " "	48.0				22090	20240	18670	17310	16080	15030	14070	13230	12480	11690	11070	10510	10000
8″ x 8″ " "	58.0					24410	22530	20880	19390	18120	17000	15990	15070	14240	13500	12810	12180
8″ x 8″ " "	67.0					28240	26090	24190	22470	20980	19650	18470	17410	16480	15610	14800	14110
10″ I	25.4					10810	9970	9240	8580	8020	7510	7060	6660	6300	5970	5670	5400
10″ I	35.0					14080	12980	12080	11180	10450	9790	9190	8670	8200	7760	7370	7010
10″ x 5¾″ W.F.	21.0					9250	8530	7900	7330	6850	6420	6040	5690	5390	5100	4850	4610
10″ x 5¾″ " "	25.0					11100	10260	9510	8840	8260	7740	7280	6860	6510	6180	5860	5580
10″ x 8″ " "	33.0					14770	13590	12610	11730	10960	10270	9650	9100	8610	8160	7750	7380
10″ x 8″ " "	39.0					17580	16220	15030	13960	13040	12230	11490	10840	10260	9720	9240	8790
10″ x 8″ " "	45.0					20360	18760	17380	16170	15100	14150	13310	12550	11870	11250	10690	10170
10″ x 10″ " "	49.0						20580	19090	17740	16570	15550	14610	13780	13030	12350	11730	11170
10″ x 10″ " "	54.0						22720	21090	19590	18300	17170	16130	15220	14390	13650	12960	12330
10″ x 10″ " "	66.0						28070	25760	23940	22350	20960	19710	18580	17570	16660	15830	15060
12″ x 8″ " "	45.0									15940	14940	14050	13260	12540	11890	11310	10750
12″ x 10″ " "	53.0									18980	17800	16740	15790	14940	14170	13450	12820
12″ x 12″ " "	65.0									23380	21940	20620	19450	18420	17470	16590	15810

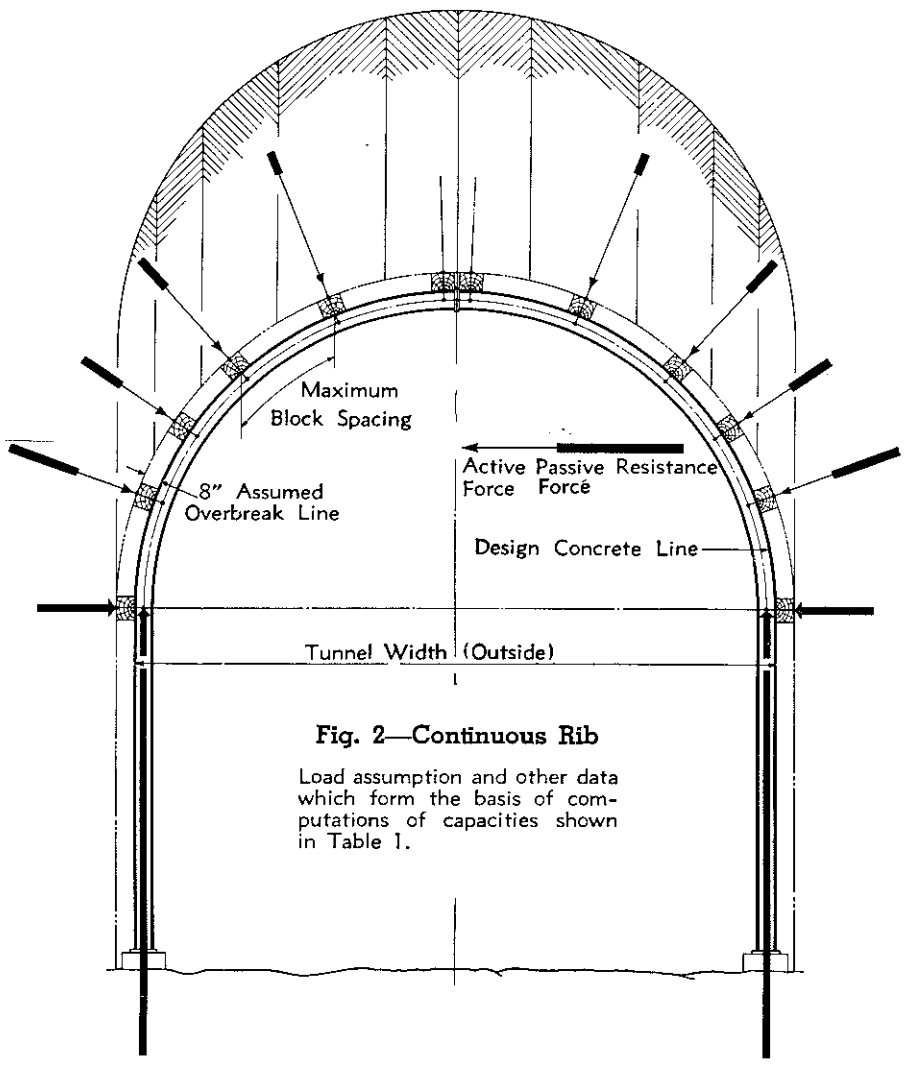


Fig. 2—Continuous Rib

Load assumption and other data which form the basis of computations of capacities shown in Table 1.

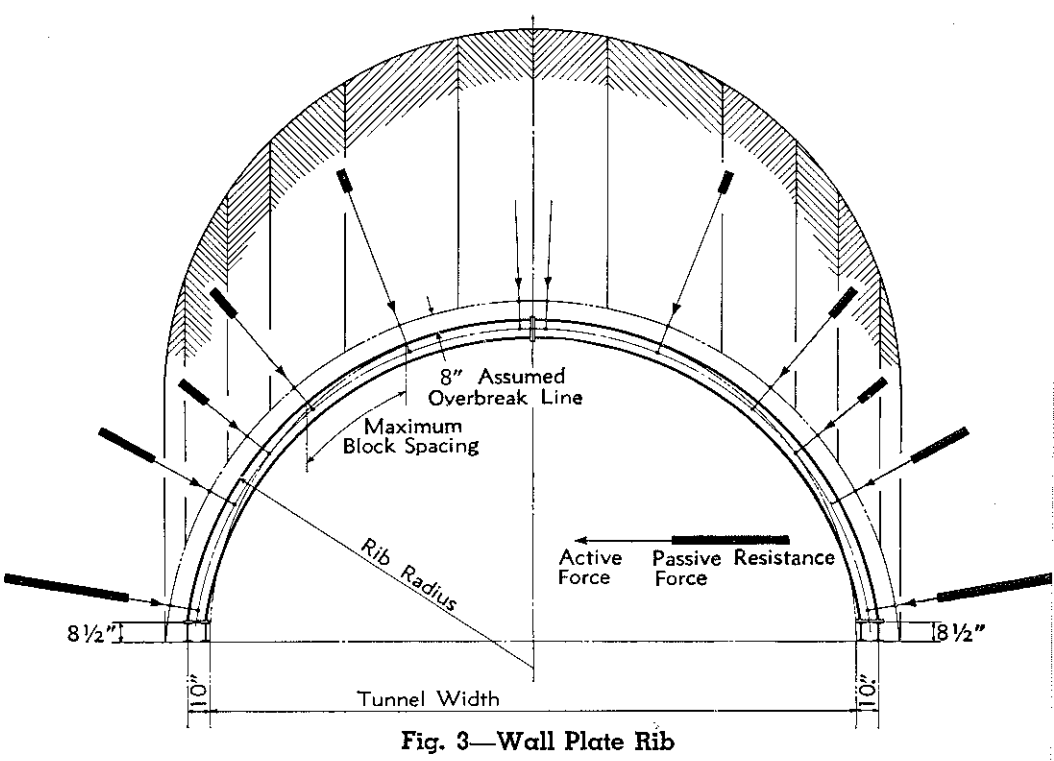


Fig. 3—Wall Plate Rib

Load assumption and other data which form the basis of computations of capacities shown in Table 2.

TABLE 2—WALL PLATE RIBS

CAPACITY IN POUNDS PER FOOT OF TUNNEL WIDTH — MAXIMUM FIBER STRESS—27000 LBS. PER SQ. IN.

BEAM		Width of Tunnel to Outside Design Concrete Line															
Nominal Depth, Flange Width & Type	Wt. Per Foot	12′-0″	14′-0″	16′-0″	18′-0″	20′-0″	22′-0″	24′-0″	26′-0″	28′-0″	30′-0″	32′-0″	34′-0″	36′-0″	38′-0″	40′-0″	42′-0″
		40″	42″	44″	46″	48″	50″	52″	54″	56″	58″	60″	62″	64″	66″	68″	70″
		Maximum Blocking Point Spacing															
4″ I	7.7	3090	2780														
4″ x 4″ H	13.0	5380	4850	4400	4020												
5″ I	10.0	4530	4070	3690	3360												
5″ x 5″ Stanchion	16.0	7780	7000	6330	5770												
5″ x 5″ H	18.9	8840	7940	7190	6550												
6″ I	12.5	6290	5660	5110	4650	4260											
6″ I	17.25	7990	7180	6490	5920	5430	5010										
6″ x 4″ Light Beam	12.0	6200	5560	5020	4570	4200											
6″ x 4″ ″ ″	16.0	8480	7600	6870	6270	5740	5300										
6″ x 6″ Stanchion	15.5	8380	7500	6780	6180	5660	5230										
6″ x 6″ H	20.0	10740	9630	8710	7930	7270	6700	6220	5780								
6″ x 6″ H	25.0	13270	11890	10770	9800	8980	8280	7680	7140	6670							
7″ I	15.3			6740	6130	5610	5190										
8″ I	18.4			8590	7820	7170	6610										
8″ I	23.0			10240	9330	8550	7890	7310	6790	6340	5940	5580	5250				
8″ x 4″ Light Beam	15.0			7110	6470	5930	5470										
8″ x 8″ H	34.3			16820	15310	14020	12940	12000	11160	10410	9750	9170	8640	8180	7740	7350	6990
8″ x 5¼″ W.F.	17.0			8220	7510	6880	6360	5890									
8″ x 5¼″ ″ ″	20.0			9820	8940	8160	7550	7010	6500	6070							
8″ x 6½″ ″ ″	24.0			11920	10860	9930	9170	8500	7900	7380	6920	6490	6120	5790	5490		
8″ x 6½″ ″ ″	28.0			14010	12670	11600	10710	9930	9220	8610	8070	7580	7150	6770	6410		
8″ x 8″ ″ ″	31.0			15550	14160	12970	11970	11090	10300	9630	9020	8470	7990	7560	7150	6790	6470
8″ x 8″ ″ ″	35.0			17590	16030	14750	13540	12550	11670	10900	10200	9600	9040	8550	8100	7680	7310
8″ x 8″ ″ ″	40.0			20100	18300	16750	15470	14330	13320	12430	11650	10960	10330	9760	9250	8770	8350
8″ x 8″ ″ ″	48.0				22090	20240	18670	17310	16080	15030	14070	13230	12480	11690	11070	10510	10000
8″ x 8″ ″ ″	58.0					24410	22530	20880	19390	18120	17000	15990	15070	14240	13500	12810	12180
8″ x 8″ ″ ″	67.0					28240	26090	24190	22470	20980	19650	18470	17410	16480	15610	14800	14110
10″ I	25.4					10810	9970	9240	8580	8020	7510	7060	6660	6300	5970	5670	5400
10″ I	35.0					14080	12980	12080	11180	10450	9790	9190	8670	8200	7760	7370	7010
10″ x 5¾″ W.F.	21.0					9250	8530	7900	7330	6850	6420	6040	5690	5390	5100	4850	4610
10″ x 5¾″ ″ ″	25.0					11100	10260	9510	8840	8260	7740	7280	6860	6510	6180	5860	5580
10″ x 8″ ″ ″	33.0					14770	13590	12610	11730	10960	10270	9650	9100	8610	8160	7750	7380
10″ x 8″ ″ ″	39.0					17580	16220	15030	13960	13040	12230	11490	10840	10260	9720	9240	8790
10″ x 8″ ″ ″	45.0					20360	18760	17380	16170	15100	14150	13310	12550	11870	11250	10690	10170
10″ x 10″ ″ ″	49.0						20580	19090	17740	16570	15550	14610	13780	13030	12350	11730	11170
10″ x 10″ ″ ″	54.0						22720	21090	19590	18300	17170	16130	15220	14390	13650	12960	12330
10″ x 10″ ″ ″	66.0						28070	25760	23940	22350	20960	19710	18580	17570	16660	15830	15060
12″ x 8″ ″ ″	45.0									15940	14940	14050	13260	12540	11890	11310	10750
12″ x 10″ ″ ″	53.0									18980	17800	16740	15790	14940	14170	13450	12820
12″ x 12″ ″ ″	65.0									23380	21940	20620	19450	18420	17470	16590	15810

Values shown are for stress of 27 Kips
For 24 Kips multiply by .889

RIB DETAILS

BUTT JOINT

When used at the crown this joint (shown in *Figs. 4 a* and *4 b*) is fabricated to be open at the top. As the rib is pre-stressed by wedging, this joint tends to close to full bearing. When used elsewhere the "Butt Joint" is fabricated for initial full bearing.

Butt Plates—

Height—

Rib depth plus 1″.

Width—

Rib flange width plus 1″, minimum 7″.

Thickness—

4″ and 5″ ribs — 3/8″
6″ and 7″ ribs — 1/2″
8″ and up ribs — 5/8″

Hole Spacing

A—3″
B—2-3/4″
C—Nominal rib depth, minus 5″

Bolt Size—

4″ and 5″ ribs — 3/4″ x 2″, sq. hd.
6″ and up ribs — 1″ x 3″, hex. hd.

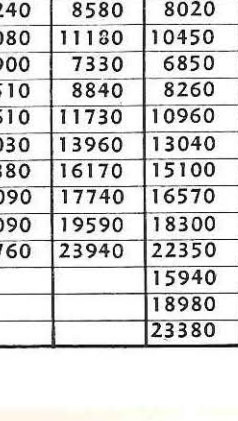


Fig. 4a 4″ to 7″ Ribs

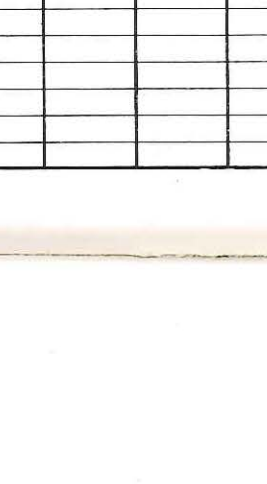


Fig. 4b 8″ and Larger Ribs

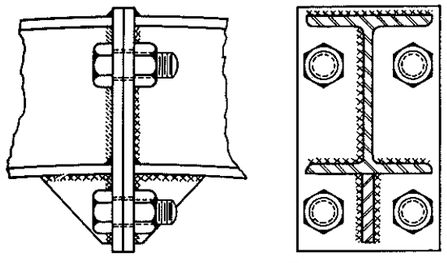
Dimensions other than Commercial's standards furnished to meet special conditions.

FLEXURE BUTT JOINT

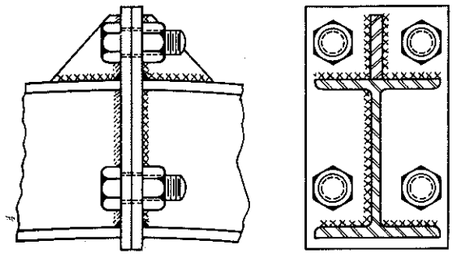
Where it is necessary that a rib joint transmit bending stress Commercial furnishes a butt joint as shown in *Fig.* 5 designed to transmit tension at either flange. This joint is far superior to the spliced joint commonly used for this purpose in other constructions because:

There is no reduction of the sectional area of the rib for bolt holes hence no reduction in the capacity of the rib.

The butt joint is fully effective when bolted whereas the spliced joint must slip until the bolt hole clearance is taken up before it is fully effective.



For Tension at Inner Flange



For Tension at Outer Flange

Fig. 5

Erection time and cost is only a fraction of that required for a spliced joint. A small number (4 or 6) large bolts displace a large number (20 to 40) of smaller bolts, some of which (web) are awkward to tighten. There are no loose pieces to get lost.

Initial cost of a butt joint is less than that of a spliced joint.

BASIS OF DESIGN—

Each "Flexure Butt Joint" is specially designed for the application involved. The joint is not designed to develop the full strength of the rib in bending as this is not necessary.

Bending moment in a rib is due only to the "Thrust" acting about the arc "rise" between blocking points as a lever arm. (See page 216.) The thrust produces compressive stress uniform over the cross section of the rib. Bending produces compressive stress in one side of the rib and tensile stress in the other. The "Flexure Butt Joint" is designed for the excess of tensile stress over the compressive stress.

TENSION BUTT JOINT

In some tunnels, particularly in circular pressure tunnels, the circular ribs are intended to carry all or part of the ring tension in the finished structure.

The initial function of the rib, during the construction stage, is to carry the external load of the surrounding rock or ground, which causes compression in the rib. Later, when water under pressure is admitted to the finished tunnel, the tension may exceed the compression already in the rib, especially in those ribs which are not fully loaded.

This reversal of stress requires a joint that will not "give" when the direction of force is changed.

Welded Joints—

Some tunnels have been built using ribs with spliced joints. After erection, and before concreting, the splice plates were field welded to prevent local "give" when the stress was reversed.

The welded joint is fully effective but its cost is high, particularly its field cost. Where bolts are used in the splice plates the erection cost is high due to the number of bolts and the necessary holes. The field welding is expensive, and in addition involves equipping for an operation not commonly required in a tunnel. Also, conditions are usually less than ideal for welding.

In some cases a combination of butt joint and welded flange splices has been used effectively, the flange splices shop welded to opposite ends of each rib segment so that half the welding is done in the shop, and only half in the field. Erection is accomplished by bolted butt joints.

Commercial has furnished "Tension Butt Joints" which fully develop the rib strength in tension (and compression) and which are pre-stressed to prevent any "give" when the loading is reversed. (fig. 6). They have the advantage of low erection cost and more importantly, the initial erection is the final erection.

Pre-stressed Design—

Each joint is specially designed to meet the conditions attending its use. Heavy butt plates are welded to the rib ends and reinforced against bending by gussets. Welds are suitably designed. Bolts, usually of high tensile steel are also proportioned as to size and number and are prestressed at erection to transmit the forces required.

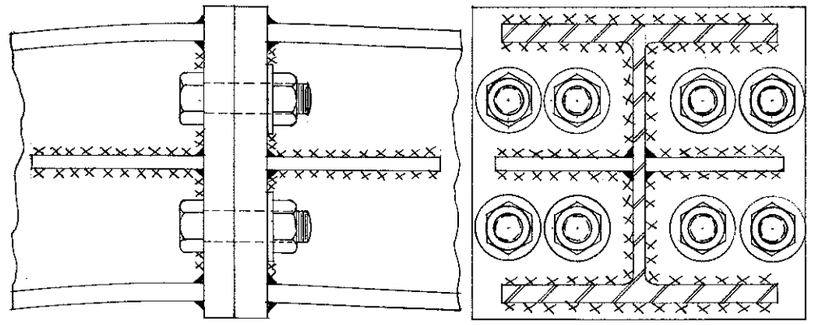


Fig. 6

SPLICED JOINTS—

While spliced joints are very seldom used in tunnel support, the butt type joints having been proven very satisfactory, Commercial can furnish any type joint of maximum efficiency.

LAGGING STOPS

Frequently on the "Continuous Rib" type of rib, a shelf angle is welded across the outer flange at spring line. Erection of lagging starts upward from this angle, or the angle may serve to support blocking at spring line. Additional stop angles are furnished as required.

LATERAL BRACING HOLES

Pairs of holes are punched in the webs of ribs for tie rods or spreaders, required for lateral bracing.

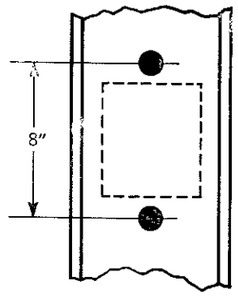


Fig. 7

Number of Holes—

Not less than three pairs in each curved rib segment, one near each end and one or more intermediate.

Not less than two pairs in each straight rib segment.

Spacing—

As required by design consideration but not more than 60 times the radius of gyration about the minor axis.

Center Distance—

For collar braces between tie rods, —8" *(Fig. 7.)*

For collar braces above tie rods, and for spreaders,—3" *(Fig. 8.)*

Location—

On center line of rib unless required elsewhere.

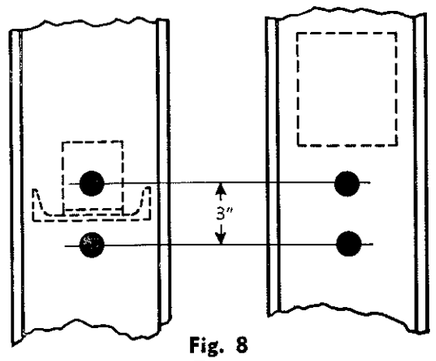


Fig. 8
Typical

Dimensions other than Commercial's standards furnished to meet special conditions.

FOOT for use on Wood Blocking

Foot Plate—

Area—

Sufficient foot plate area is provided to distribute the rated capacity of the rib over the blocking at a maximum of 1000 lbs. per sq. in.

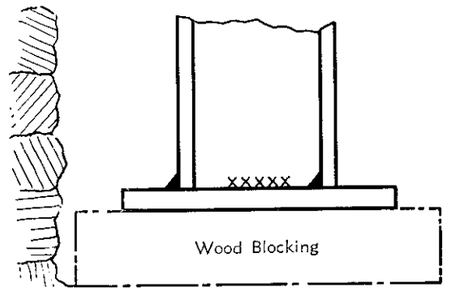


Fig. 9

Standard Foot Plates for use with various size ribs.

Rib Size	Foot Plate Size
4" I	— ½" x 6" x 7"
5" I	— ½" x 7" x 8"
6" I	— ½" x 7" x 9"
8" I	— ⅝" x 8" x 10"
4" H	— ½" x 8" x 8"
5" H or CB	— ⅝" x 9" x 9"
6" H or CB	— ⅝" x 10" x 10"
8" x 6½" WF	— ⅝" x 10" x 12"
8" x 8" WF	— ¾" x 12" x 12"
10" x 8" WF	— ¾" x 12" x 14"
10" x 10" WF	— ¾" x 16" x 16"

DOUBLE FOOT

A rock tunnel is usually holed through before concreting which means that loads have a long time to build up. When doubt exists as to the ability of the ground under the foot blocks, or the foot blocks themselves, to carry the increasing loads, it is desirable to pour the invert concrete soon after the steel support is erected.

The "Double Foot," shown in *Fig. 10,* first puts the load on the foot blocks and later, on the invert concrete, when poured.

Foot Plate—

Designed as above for wood blocking.

Invert Plate—

Area—

Is sufficient to spread the rated capacity of the rib over the concrete at 1200 lbs. per sq. in.

Elevation—

Is such that the upper face is flush with the top of the invert concrete, except where it is desired to set the ribs "high." In this case the invert plate is attached lower.

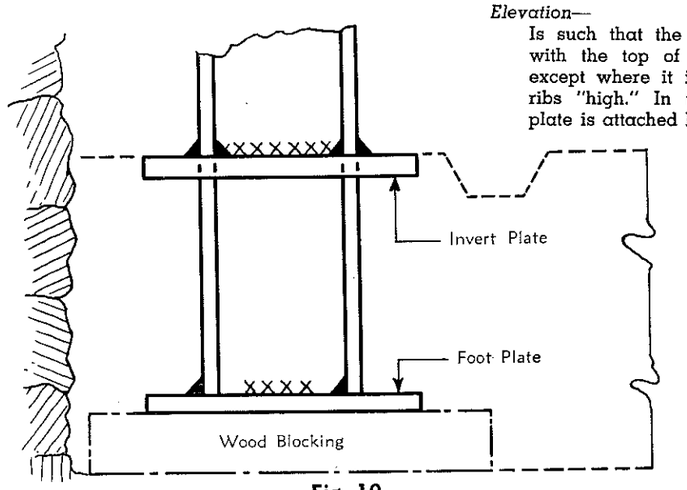


Fig. 10

RIB DETAILS

FOOT for use on Double Beam Wall Plate

Foot Plate—

Length—

Out to out of wall plate beam flanges.

Width—

Rib flange width plus 1/2", minimum 7"

Thickness—

1/2"

Hole Spacing "A"—

4" to 7" Ribs — 4-1/2"
8" and up Ribs — 5-1/2"

Thrust Cleat—

5/16" x 3-1/4" x 3-1/4"

Toggle Plates—

Length—

6-1/2"

Width—

4" to 7" Ribs — 3"
8" and up Ribs — 4"

Thickness—

1"

Bolts—

1" x 3"

Blocking Plate—

Furnished on request.

Height—

8" or as specified

Length—

8" or as specified

Thickness—

3/8"

Dimensions other than Commercial's standards furnished to meet special conditions.

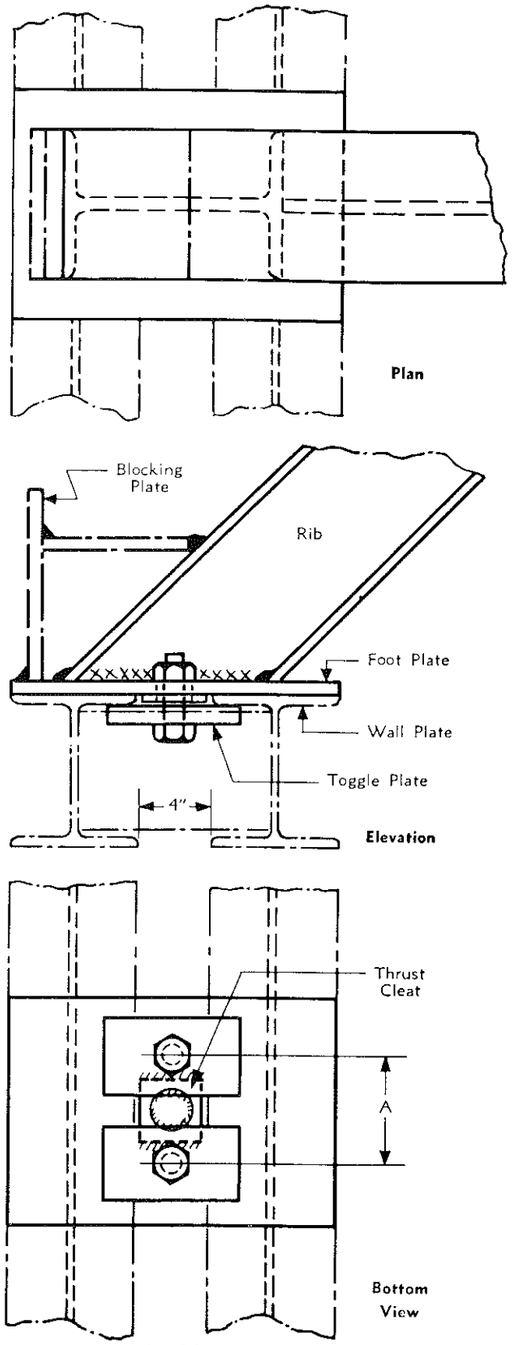
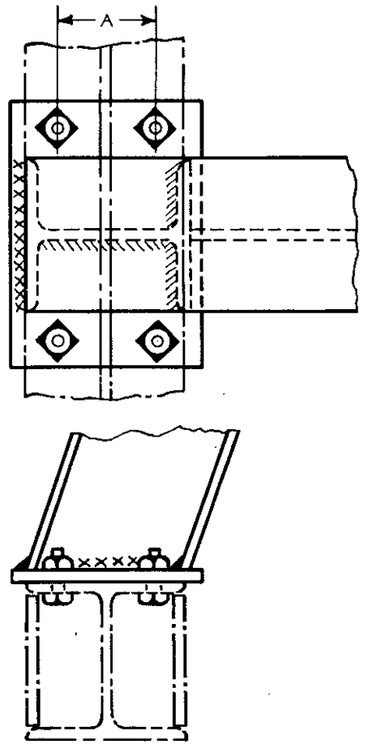


Fig. 11

RIB DETAILS



Bolts—

3/4" x 2"

Blocking Plate—

Furnished on request.

Height—

8" or as requested.

Length—

8" or as requested.

Thickness—

3/8"

Dimensions other than Commercial's standards furnished to meet special conditions.

FOOT for use on Single Beam Wall Plate.

Foot Plate—

Length—

Wall beam flange width or rib depth on the mitre, (whichever is the greater) plus 1".

Width—

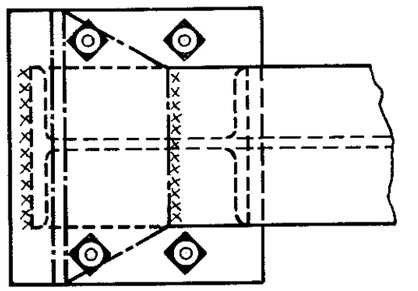
Rib flange width plus 5-1/2"

Thickness—

1/2"

Hole Spacing "A"—

For 6" Wall beam flange — 3-1/2"
For 8" Wall beam flange — 5"
For 10" Wall beam flange — 5"



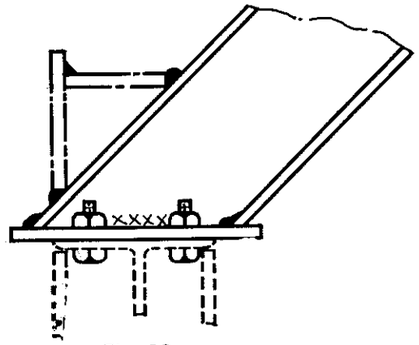


Fig. 13

FOOT for use on Flat Wall Plate

Most frequently, when using a "Flat" wall plate, no foot is required for the rib.

In some cases it is desirable to align ribs individually by wedging between rib and wall plate, in which event a foot is provided on the rib. (See *Fig. 14.*)

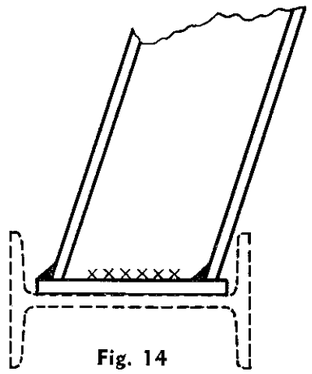


Fig. 14

Foot Plate—

Length—

Distance between inside of flanges of the "Flat" wall plate, minus 1/2" for clearance.

Width—

Rib flange width plus 1/2".

Thickness—

Usually 3/8".

Holes—

None.

SLIP-ON FOOT

If it is anticipated that the length of the leg may vary, a loose foot is provided which is slipped on after the leg has been burned off in the field to suit the condition encountered.

Two angles are welded to the foot plate to locate the foot on the leg, as shown in *Fig. 15.*

Foot Plate—

Same as "Foot, for use on Wood Blocking," see page 245.

Locating Angles—

Size—

2-1/2" x 2-1/2" x 1/4".

Length—

Distance between inside of rib flanges less 1" for clearance and weld.

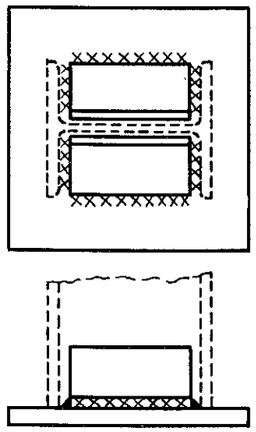


Fig. 15

RIB FOOT for use on Posts

Foot Plate—

Length—

Rib or post depth on mitre (whichever is greater) plus 1".

Width—

Rib or post flange width (whichever is greater) plus 1/2", minimum 7".

Thickness—

4" to 7" ribs — 1/2"
8" and up ribs — 5/8"

Hole Spacing—

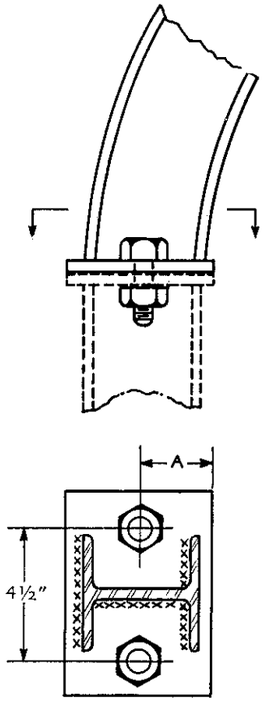
"A"—3"

"B"—2-3/4"

"C"—Nominal Rib or Post depth (whichever is less) minus 5".

Bolt Size—

4" and 5" Ribs—3/4" x 2" sq.
6" and up Ribs—1" x 3" hex.



4" to 7" Ribs

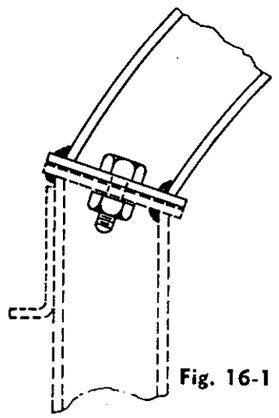
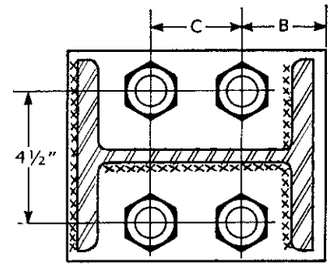


Fig. 16-1

Dimensions other than Commercial's standards furnished to meet special conditions.



8" and Larger Ribs

Fig. 16

RIB DETAILS

WALL PLATE BRACKET

When using the "Rib and Post" type of support system, it may become desirable or necessary to carry the roof ribs on a wall plate set in a hitch, until the bench can be removed and posts set.

This contingency is met by the use of the "Wall Plate Bracket" welded to the ribs as shown in *Fig. 17.*

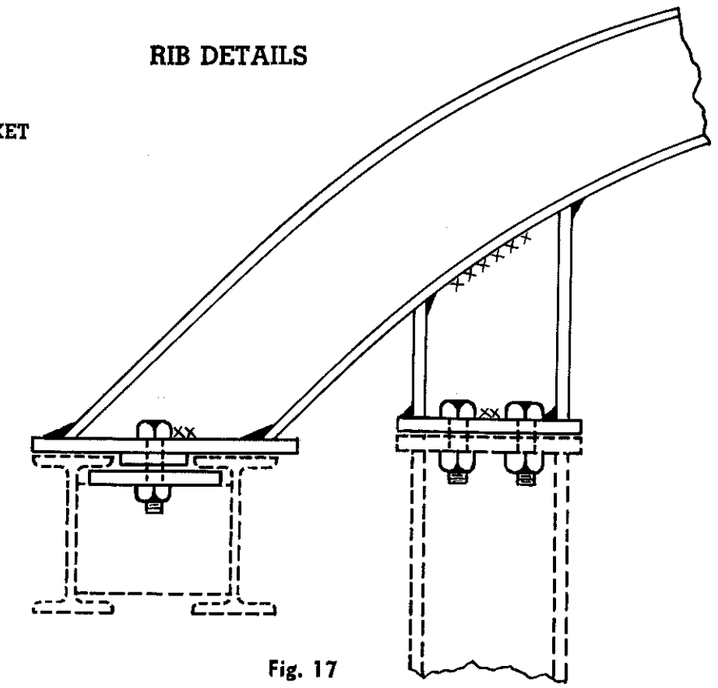


Fig. 17

TRUSS PANEL LUGS

If the operation involves the use of "Truss Panels" to bridge the bench shots, lugs are welded to the ribs for attachment of the panels. See *Fig. 18.*

Each application is the subject of a special design study.

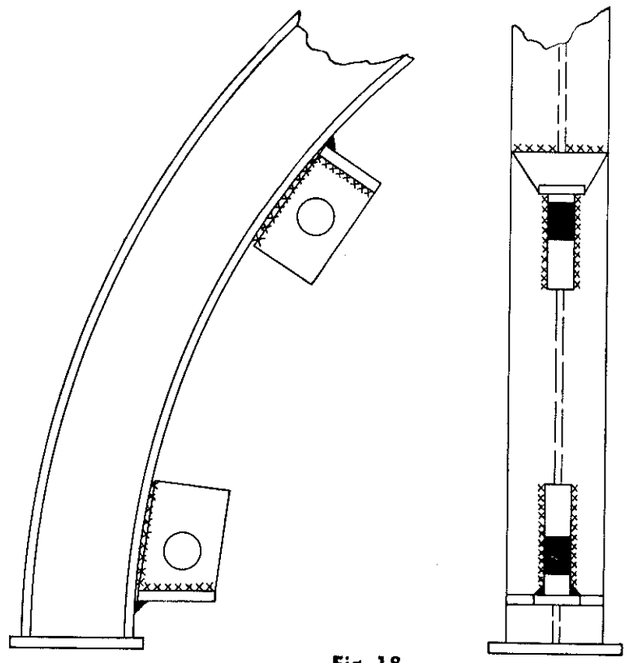


Fig. 18

POST DETAILS

POST

Posts are designed as columns with restrained ends to carry rib loads, both active and passive. Fibre stresses are reduced in consideration of the slenderness ratio about the major axis.

Lateral Bracing Holes—

Same as for ribs, see page 244.

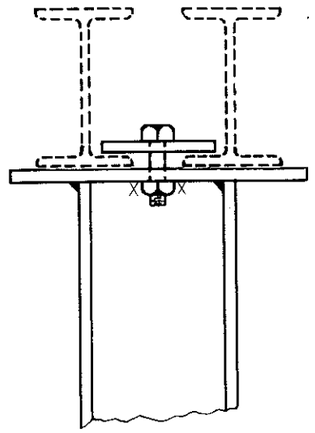


Fig. 19

Foot—Same as for ribs, see page 245.

Cap—Used with

Double Beam Wall Plate—
Same as rib foot except no thrust cleat is required. See page 246.

Single Beam Wall Plate—
Same as rib foot. See page 247.

Flat Wall Plate—
Same as rib foot, when required. See page 248.

Rib and Post Type—
To meet requirements of rib foot. See page 249.

Blocking Plate—

When the post is a part of the "Rib and Post" type of support system and the arch is relatively flat, it is sometimes desirable to block the top of the post.

An angle blocking plate may be furnished as shown at "A," *Fig. 20,* or a shelf angle as shown at "B" may be provided if the post flange is wide enough to block against.

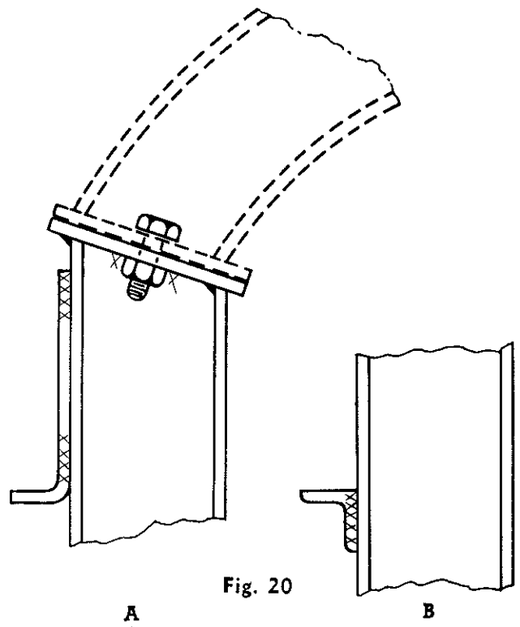


Fig. 20

A B

STRUT DETAILS

INVERT STRUTS

When side pressures develop and no prior provision has been made to convert the section to a full circle, struts across the invert are commonly used. They provide horizontal reaction at the foot of the ribs. Also, they distribute part or all of the vertical and horizontal loads to the bottom, thereby holding it down.

Struts are usually made of the same beam section used in the rib. The curvature conforms to the bottom of the tunnel section. If the bottom is straight, the strut should nevertheless be given some curvature as this will place the steel in compression rather than bending.

Connection with the rib may be as shown in *Fig. 21*, or any other convenient way.

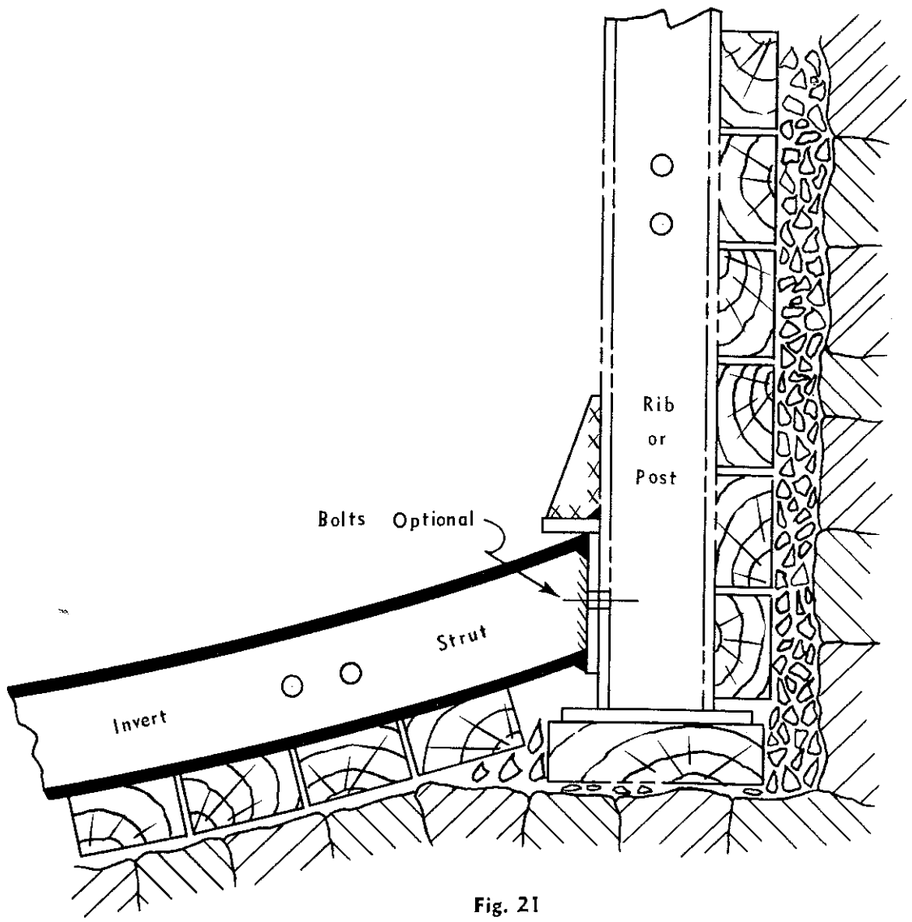


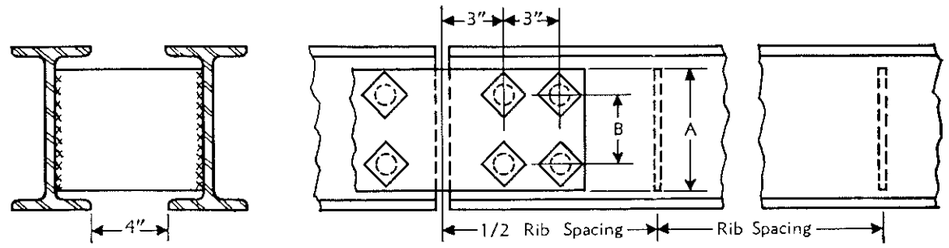
Fig. 21

WALL PLATE DETAILS

DOUBLE BEAM WALL PLATE

Wall plates of two I-beams as shown in *Fig. 22* have become standard in rock tunnels. Their broad surfaces are ideal for blocking. Their box section resists twist. Their wide base reduces tendency to roll.

Ribs and posts are clamped—no bolt holes to match up. This feature speeds erection and permits variation in both rib and post spacing, an important advantage in many cases where loads are heavier than anticipated.



Beams—

Usually I-beams, but can be made out of channels or combination of a channel and I-beam.

Length—

Furnished in any desired multiple of rib spacing.

Diaphragms

One under each rib location

Thickness—

6″ and 7″ beam depth—3/8″
8″ and up beam depth—1/2″

Splices—

Two on each web; 4 per Wall Plate

Length—

15″

Thickness—

1/4″

Width and Vertical Hole Spacing

	Width "A"	*Hole Spacing "B"*
6″ beam depth	4-1/4″	2-3/4″
7″ beam depth	5-1/4″	3-1/2″
8″ beam depth	6″	3-1/2″
10″ beam depth	7-1/2″	5″
12″ beam depth	9-1/2″	7″

Bolts—

16—3/4″ x 2″ per Wall Plate

Dimensions other than Commercial's standards furnished to meet special conditions.

SINGLE BEAM WALL PLATE

This type has been largely superseded by the "Double" which provides wider blocking surface, greater resistance to twisting and "rolling," quicker attachment of ribs and posts, and is cheaper both in first cost and erected cost.

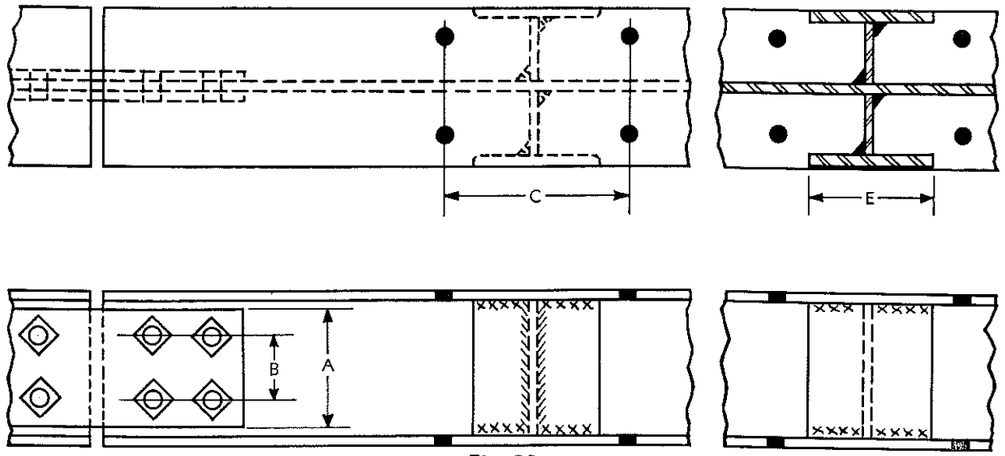


Fig. 23

Beam—

Any H-beam whose flange width is equal to or greater than the rib or post depth. Its strength must satisfy the loading assumption.

Length—

Furnished in any desired multiple of rib spacing.

Hole Spacing "C"—

Rib flange width plus 3″

Hole Spacing "D"

For 6″ wall plate flange width—3-1/2″
For 8″ wall plate flange width—5″
For 10″ wall plate flange width—5″

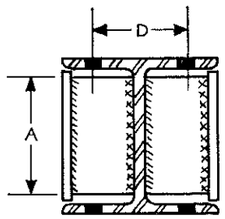


Fig. 24

Tee Stiffeners—

One under each rib on each side.

Flange—

Thickness and width "E", *Fig. 23,* approximately equal to rib flange dimensions.

Web—

Thickness 1/4″—Height "A", *Fig. 24,* same as splice plate width.

Splices and Bolts—

Same as for "Double" beam wall plate. See page 253.

Dimensions other than Commercial's standards furnished to meet special conditions.

FLAT WALL PLATE

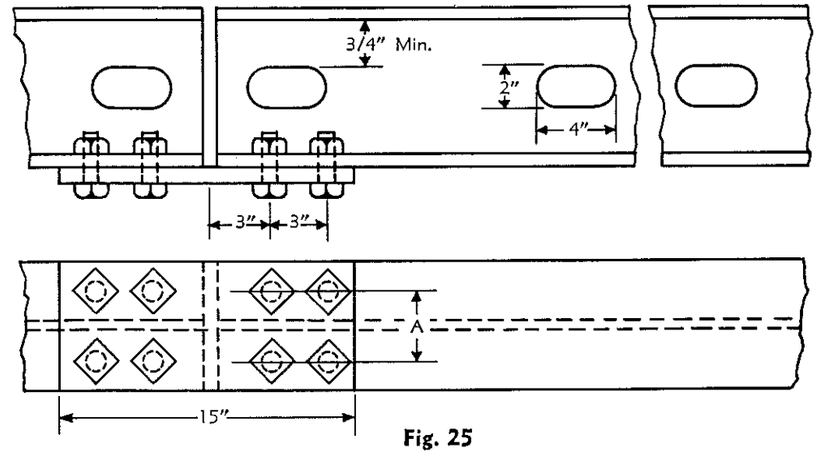


Fig. 25

Beam Section—

Usually a "Wide Flange" beam which will just admit the rib and post. The minimum flange width and weight are usually used.

Length—

Furnished in any desired multiple of rib spacing.

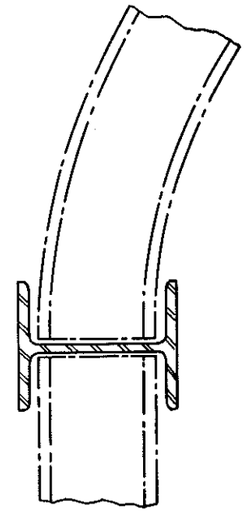


Fig. 26

Vent Holes—

Spacing—

About 12″ centers, but some even fraction of rib spacing. If reinforcing rods are to pass through the Wall Plate, vent holes are located to suit.

Splices—

Flange splice on inside flange. Also on outside flange if desired.

Thickness—

Approximately same as flange, minimum 3/8″.

Spacing "A"—

Flange width	5-1/4″ — A is 2-3/4″
	5-3/4″ — A is 2-3/4″
	6-1/2″ — A is 3-1/2″
	8″ — A is 5″

Dimensions other than Commercial's standards furnished to meet special conditions.

CROWN BAR and HANGER DETAILS

CROWN BARS

"Double Channel" Crown Bars are designed to act as cantilevers without twisting. The torsion plates together with the diaphragms welded between the channels constitute a box section, rigid against the twisting loads from hurried blocking. The width is greater than the depth to eliminate rolling. The bottom is dished to seat properly on curved ribs. See *Fig. 27.*

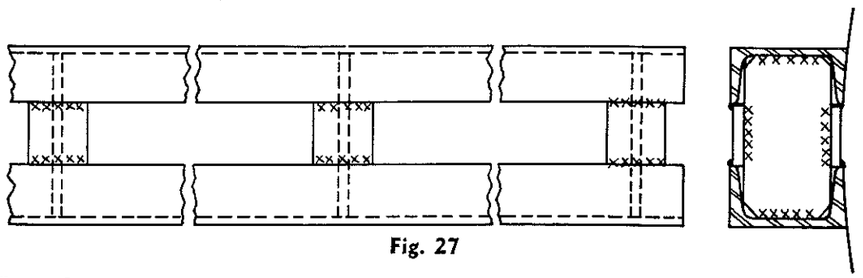


Fig. 27

Channels—

Any desired rolled channel, car, ship, or structural channels may be used. Any beam section or a combination of beam and channel may also be used.

Length—

Any length required.

Torsion Plates and Diaphragms—

Spaced approximately at 6 times the channel depth.

Plain Crown Bars—

Can be furnished in any desired H-beam section.

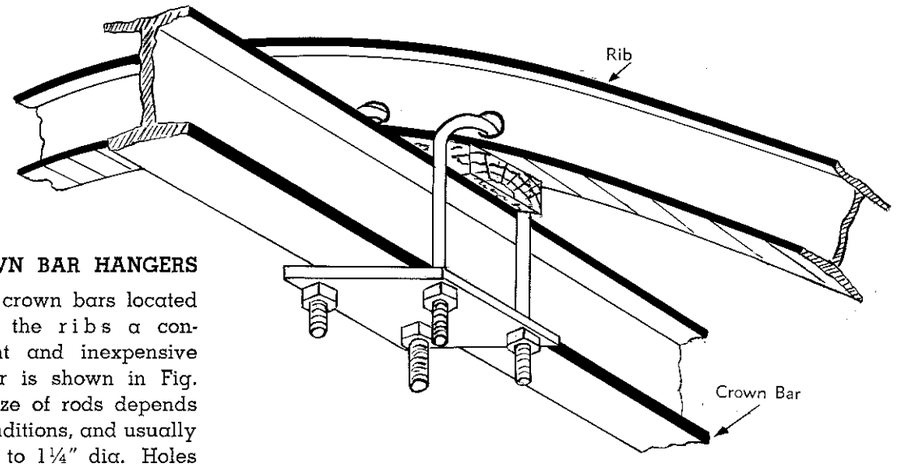


Fig. 28

CROWN BAR HANGERS

For crown bars located under the ribs a convenient and inexpensive hanger is shown in Fig. 28. Size of rods depends on conditions, and usually is ¾" to 1¼" dia. Holes must be provided in the web of ribs.

TRUSS PANEL DETAILS

TRUSS PANEL

Design—

Each set of "Truss Panels" is designed for the circumstances attending its contemplated use. They may be designed to develop the full strength of the rib, although this provision is usually not necessary as full loads take some time to develop. Commonly, 50% of the rib capacity is considered sufficient.

The span may be set at the length of the bench shot plus one rib space or preferably at double the length of the bench shot. This keeps the post further back from the face thus reducing the danger of damage.

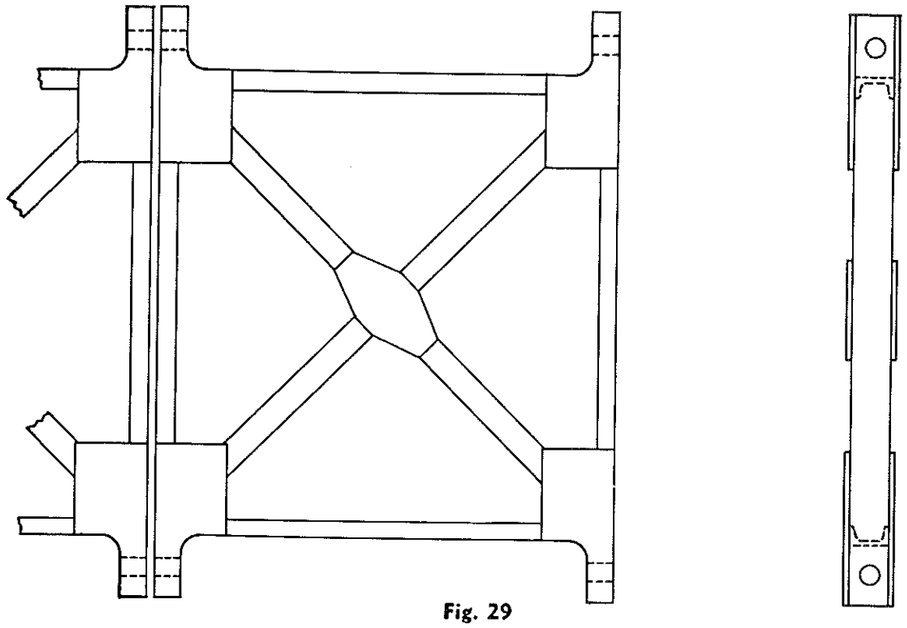


Fig. 29

Bolts—

The bolts are proportioned to develop the strength required by the design both in tension and bearing. Heat treated, high tensile steel is used, if necessary.

Rib Lugs—

Proper means of attachment to the ribs is provided in the form of lugs, proportioned to satisfy all design requirements.

BRACING DETAILS

TIE RODS—Rib Spacing Fixed—(See *Fig. 87*, page 154.)

	Tension for use with Collar Braces	*Compression*
Diameter	3/4″ or 5/8″	1″ or 7/8″
Length	Rib spacing, plus 3″	Rib spacing, plus 3″
Thread	Quick Acting, 3″ on each end	Quick Acting—3″ one end, 6″ other end
Nuts	2—square	4—hex.

TIE RODS—Rib Spacing Variable

The above rods can be furnished in random mill lengths, 15′ to 25′, to be cut off and threaded on the job. Nuts are supplied as required.

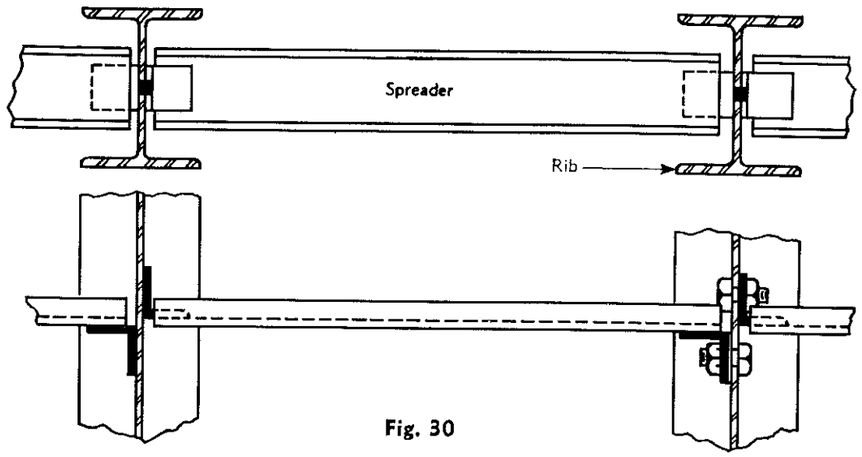


Fig. 30

SPREADERS *(Fig. 30.)*

Profile—

Any small channel, I-beam, or angle.

Length—

Rib spacing minus web thickness, minus 1/16″ for fit up "growth."

Clip Angles—

Size—

2-1/2″ x 2-1/2″ x 3/8″

Length—

To suit strut section.

Bolts—

2—3/4″ x 2″

Dimensions other than Commercial's standards furnished to meet special conditions.

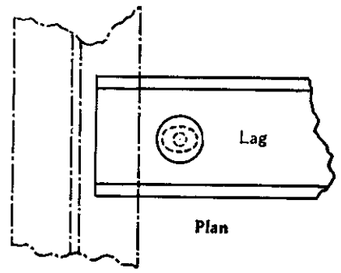
LAGGING DETAILS

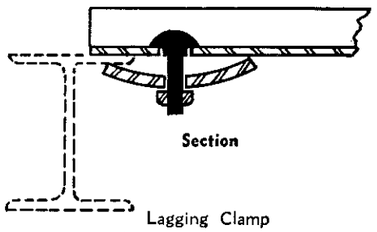
LAGGING CLAMP

The time saving, handy attachment for skeleton or tight lagging.

Applications—

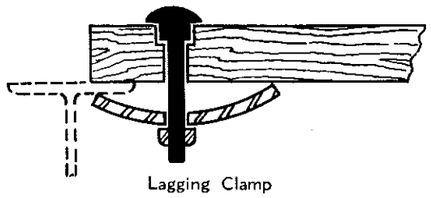
Wood	Lagging
Channel	Lagging
Beam and Plate	Lagging
Beam	Lagging
Purlin-Plate	Lagging
Water Bar	Lagging



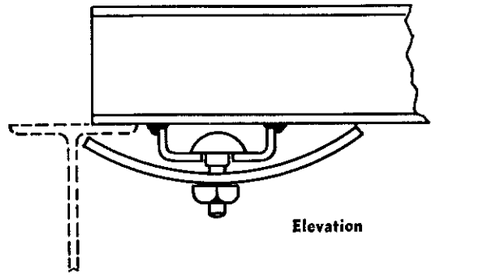


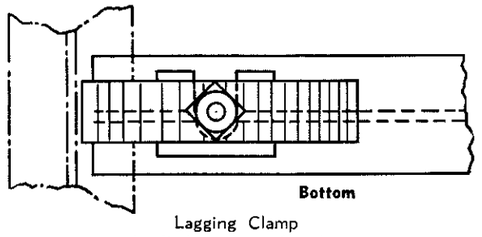
Lagging Clamp
Applied to Channel Lagging

Fig. 31



Lagging Clamp
Applied to Wood Lagging





Lagging Clamp
Applied to Beam Lagging

Fig. 32

Assembly—

When ordered with lagging, each lag is shipped with clamps and bolts assembled, nuts finger tight.

Lagging clamps and bolts are also shipped separately for assembly in the field.

Clamp—

3″ x 5″ x 3/8″.
Weight 1-1/2 lb.

Bolts—

3/4″ Carriage or track type, quick acting thread, square nut.

Length—

To suit conditions.

Dimensions other than Commercial's standards furnished to suit special conditions.

CHANNEL LAGGING

Fig. 33

Structural Steel—

Any rolled channel, but most commonly 6″ standard. *Fig. 33.*

Pressed Steel—

Fig. 34

Any desired combination of thickness, width, flange depth, and flange return to give required capacity. *Fig. 34.* Also see *Fig. 90, page 157.*

BEAM LAGGING

Fig. 35

Furnished in any desired size beam, but usually 4″ or 5″ H-beams. *Fig. 35.*

Note: The above types of lagging furnished with or without lagging clamps, as desired.

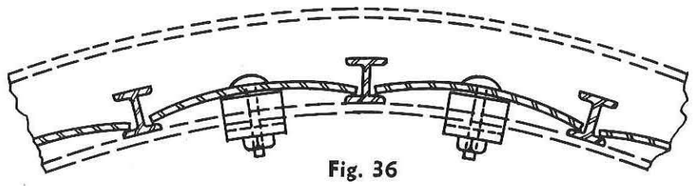


Fig. 36

BEAM AND PLATE LAGGING *(Fig. 36.)*

Beam—

Any desired size, usually 3″, 4″, or 5″ I-beams.

Plate—

Any desired size and thickness required to meet loading conditions and rib spacing.

Clamps

Lagging clamps usually furnished as they provide the ideal attachment.

LINER PLATES

Commercial Liner Plates make the ideal tight lagging where it is desired to fill the overbreak with dry pack, gravel, or concrete.

The closed corners of these plates make a skin tight enough to hold grout at low pressure or concrete wet enough to be placed by machine.

Application—

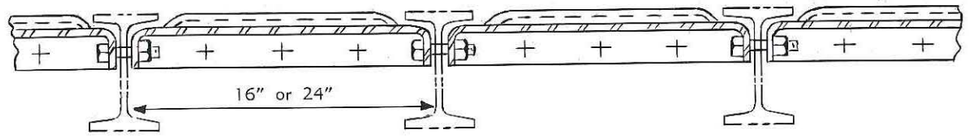


Fig. 37

When used as in *Fig. 37,* stiffener ribs run transverse of the plate (lengthwise of the tunnel). Plates are curved lengthwise to fit contour of tunnel.

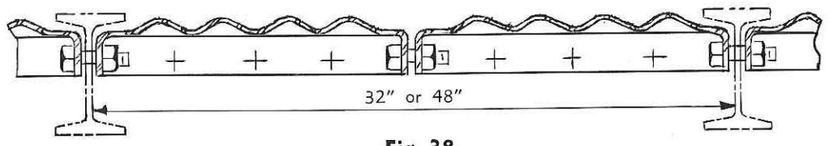


Fig. 38

When used as in *Fig. 38,* stiffener ribs run lengthwise of the plate (transverse of the tunnel). Plates are curved lengthwise to fit contour of tunnel and joints are staggered.

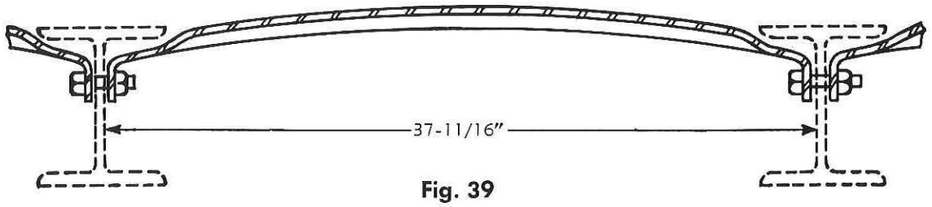
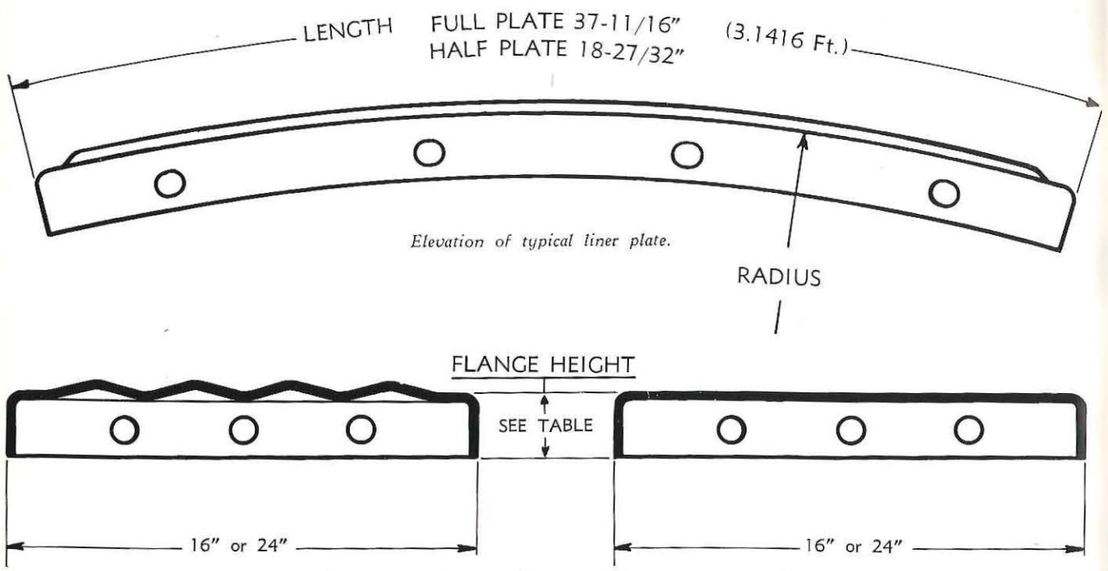


Fig. 39

When used as in *Fig. 39,* stiffener ribs run lengthwise of the plates and tunnel. Plates are arched lengthwise and curved transversely to fit contour of tunnel.

Lateral Bracing is eliminated in the plated portion of the support system.



Elevation of typical liner plate.

Cross Section of liner plates. May be furnished corrugated or plain as shown.

37-11/16" FULL SIZE AND HALF SIZE PLATES AVAILABLE

Thickness	Min. Flange Height		Weight 16" Wide Plates		Weight 24" Wide Plates		Bolt Size & Weight Pls. Above Ribs		Pls. Between Ribs	
	Side	End	Full Plate	Half Plate	Full Plate	Half Plate	Size	Weight Each	Size	Weight Each
1/8"	2"	2⅛"	27.7	14.7	39.3	21.4	½" x 1"	.17	½" x 1½"	.20
3/16"	2⅛"	2"	40.9	21.7	56.9	30.7	½" x 1½"	.20	½" x 1½"	.20
1/4"	2¼"	2⅛"	54.9	28.9	80.2	43.2	⅝" x 1½"	.36	⅝" x 2"	.36
5/16"	2⅜"	2⅛"	68.6	36.1	101.6	54.0	⅝" x 1½"	.36	⅝" x 2"	.36
3/8"	2½"	2¼"	82.3	43.3	121.9	65.8	⅝" x 1½"	.36	⅝" x 2"	.36

Bolts furnished per plate 7 5 7 5

Quick acting thread. Head and nut same size.

Grout holes Plain for 1½" or 2" pipe (1½" pipe standard)
Tapped for 1½" or 2" pipe (in 3/16" and heavier plates)

Pipe plugs for tapped grout holes supplied if requested.

PURLIN PLATE LAGGING

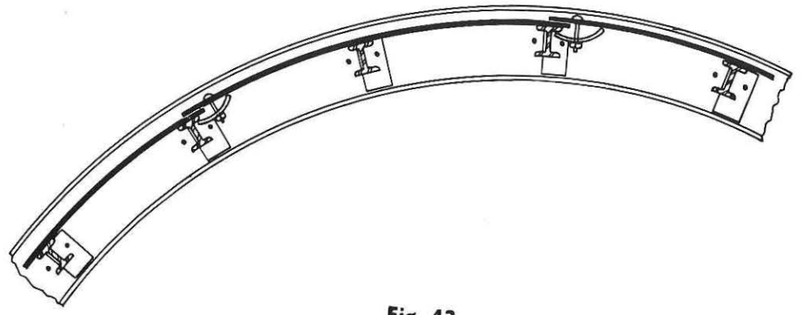


Fig. 42

Purlins—

Beam—

Any small I-beam usually 3" or 4", or channel, usually 4" or 5".

Spacing—

As required by load and lagging plate but not greater than 60 times the radius of gyration about the minor axis of the rib. The purlins act as lateral bracing for the ribs.

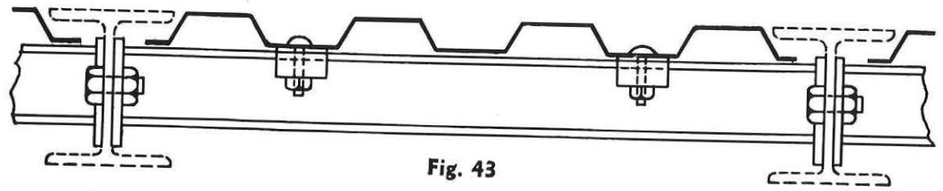


Fig. 43

Lagging Plate—

Type—

Ribbed, corrugated or flat, as desired.

Thickness—

To suit loading conditions.

Width—

To fill space between rib flanges. May be divided into two or more pieces to keep down size of individual plates.

Length—

As required, up to 10 feet.

Attachment—

By lagging clamps with carriage or track bolts.

LAGGING DETAILS

WATER BAR LAGGING

This type of "Purlin Plate" lagging is designed to shed water, diverting it down to spring line where it can be drained away.

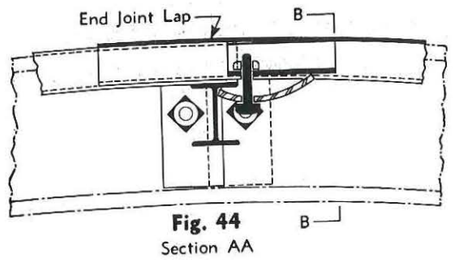


Fig. 44
Section AA

Generous laps at the end joints insure against inflow. Stiffeners are of proper depth to carry loads intended and provide ample lap on the sides.

Caution: A coarse free draining dry pack must be used to avoid trapping the water.

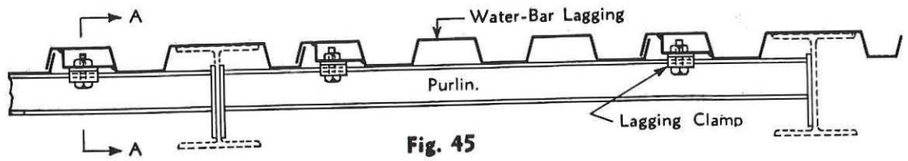


Fig. 45

Water Bar Lagging Plate—

Stiffeners—

Width and depth (not less than 2″) to suit loading requirements and rib spacing.

Thickness—

As required.

Length—

Up to 10 feet.

End Laps—

To give vertical rise of not less than 2″.

Attachment—

By lagging clamps as shown. No holes extend through the lagging plate which would permit passage of water.

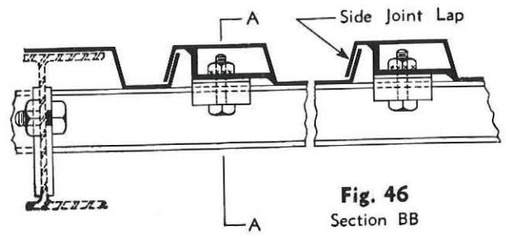


Fig. 46
Section BB

COMPATIBLE TUNNELING METHODS

Tunnel supports shown in this book may be used with all known methods of advancing rock tunnels, including the time-honored methods of drilling and blasting, and the recently developed boring procedures using various types of machines and rock cutting tools. Ordinarily, a machine would be used only where there is enough tunnel of a size and shape to justify its cost, but there have been a good many machines built in the past few years and one of suitable size and shape may be available for a shorter job.

SPECIAL SECTIONS

While the major footage of most tunnel jobs is of similar cross section, nearly all of them have areas of varying cross section or an intersection. Transitions often occur at and near the portals or at a change in tunnel size or shape. Intersections may be at a shaft or with another tunnel. Such variations usually are dictated by functional requirements. Other variations in cross section may be required by operational needs of the contractor, such as a passing track around a shaft cage, or to provide for a car dumper or some other purpose. Support for some of these various requirements are shown in Figures 62, 89, and Fig. 1, on pages 126, 156 and 237, respectively, and in Figures 1-A and 2-A following. Design of support for special sections usually requires a high degree of skill and experience as well as technical knowledge. This is especially so when the intersection or transition occurs in an area where geological conditions are unfavorable. In such cases the design must take into consideration a precedure by which the excavation can be made and supported.

Figures 1-A and 2-A show two entirely different special types of support. Both of these are very large tunnels but similar conditions occur in smaller jobs. Both of these jobs are quite well known which is one reason for selecting these particular ones for illustration.

Figure 1-A

Complicated support where Morning Glory Spillway intersects tunnel at Del Valle Dam near Livermore, California. The view is looking up the shaft from the elbow downstream. This is also a gate chamber which required a transition in the tuunel upstream and a chamber across the intersection back of the elbow which connects to an access tunnel 12 ft. diameter at the side. Also involved were air vents, etc. down the shaft.

Driven Dimensions: Bottom chamber roughly 36 ft. wide by 49 ft. high. Length of transition slightly over 130 ft. Vertical spillway 35 ft. diameter tapering out to 57 ft. diameter at the top with a 9'-6" wide pipe slot running vertically on the downstream side. Total depth approx. 138 ft. The 35 ft. diameter elbow has a radius of 60 ft. to the center.

Supports: Varied from 33" WF @ 220# to 4" H @ 13#. In the tunnel 10" WF @ 66# to 10" WF @ 49# and 8" WF @ 48# were commonly used. Main members in the shaft were generally 10" WF @ 66#.

Contractor: Green Construction Company and Winston Brothers Co., Joint venture.

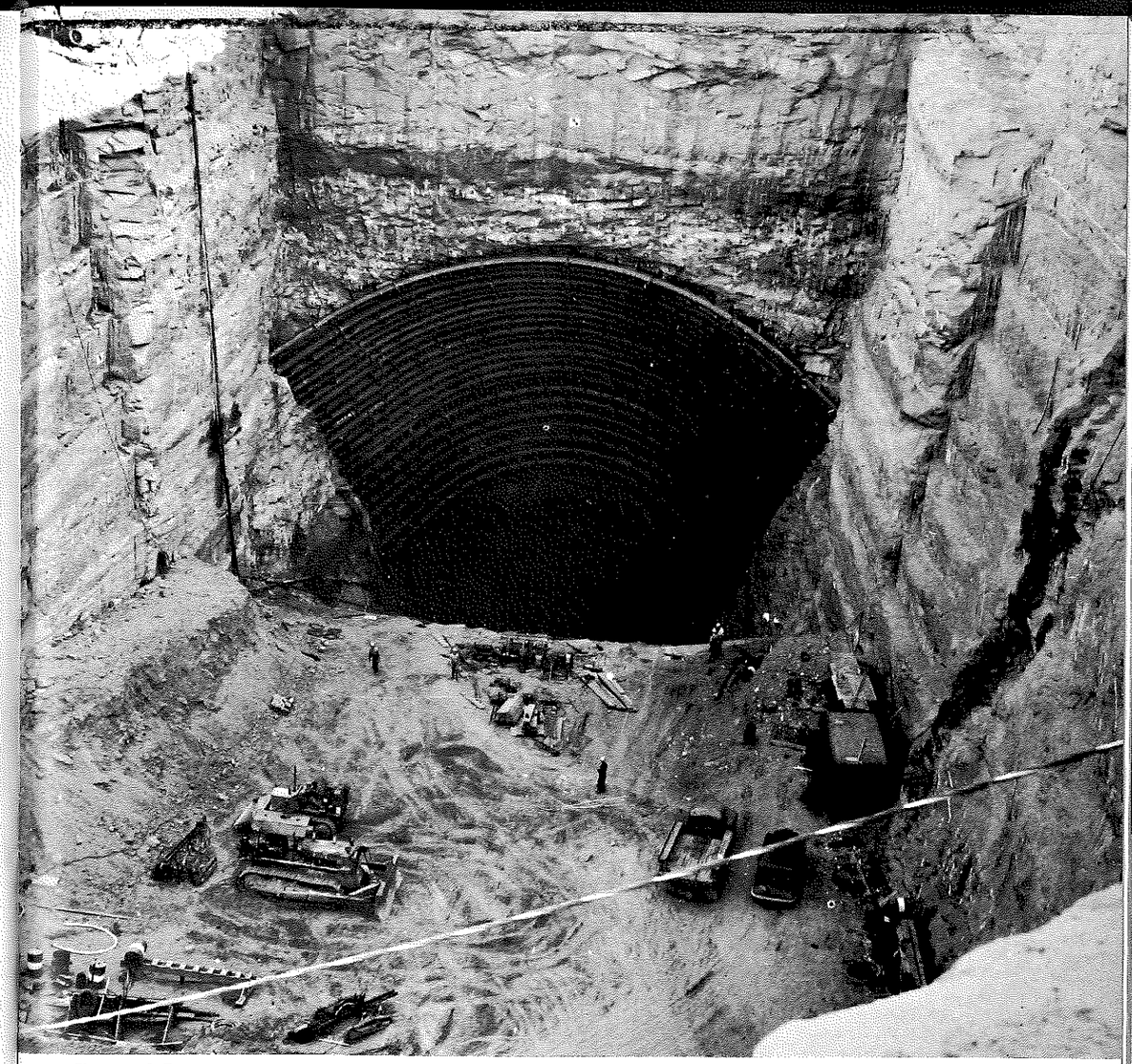


Figure 2-A

Support at portal of one spillway tunnel at Glen Canyon Dam in Northern Arizona. There are two very similar, one each side of the canyon. Size and shape change along a vertical curve from arched crown with straight sides and flat bottom to circular. Driven size: At portal, 115 ft. wide by 70 ft. high, arch supports projecting about five feet beyond side walls. At beginning of circular section 65 ft. wide by 60 ft. high.

Finished size: At portal 88'-6" wide by 52 ft. high. At start of circular portion 48'-3" I.D. reducing along 55° slope for several hundred feet to 45' I.D. where a vertical curve leads into horizontal.

Supports: Portal area 14" WF @ 167# reducing in several stages to 12" WF @ 72# at the cone. Spacing varied from 4 ft. to 6 ft. centers.

All ribs were set normal to tunnel slope and were collar braced in groups of 3 to 5 sets. Diagonal struts transmitted longitudinal forces of each group into surrounding rock to avoid accumulation along the slope.

All supports were roof arch only, hitched into rock except wider sets rested on individual grillages of 12" WF @ 92#.

Contractor: Merritt-Chapman & Scott Corp., General Contractor — Sub-Contractors on spillway tunnels Frazier-Davis Construction Co. and Northwood Construction Co.

ROOF AND SHAFT SUPPORTS IN MINES

Commercial supplies various types of support for roof and ground control in mines. Included are the regular type of fixed joint arch and ring supports commonly used in construction of water and sewer tunnels, highway and railroad tunnels, and other tunnels and shafts where the cross section must be maintained. These are sometimes referred to in mine parlance as Rigid Supports.

Also available are Commercial T-H yieldable arches for use where ground pressure changes are severe and some changes in the tunnel cross section can be tolerated. These supports have the capability of adjusting to the ground behavior, permitting some ground movement with a yielding resistance. Response of these supports to varying and unpredictable ground loads is well known, especially in Europe where this type of support was developed.

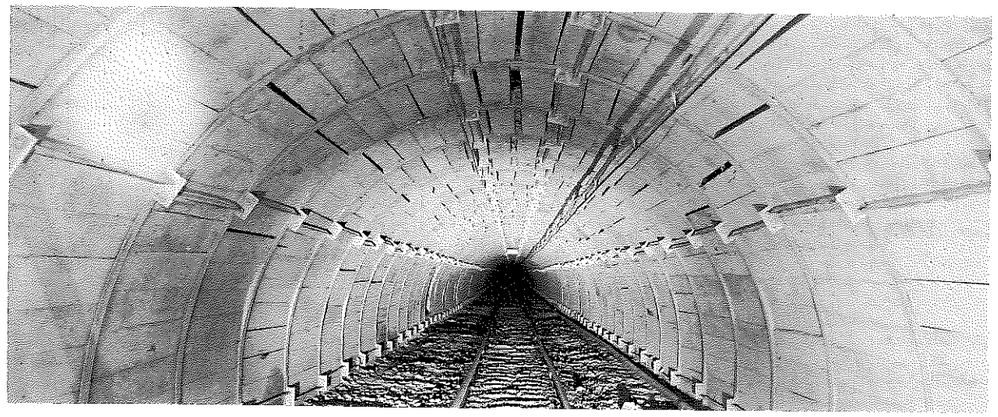
Commercial also supplies steel tunnel liner plates, either as complete support structures or as lagging for use with steel arch ribs or rings. Liner plates and various other forms of lagging are described on the foregoing pages, except for a special form of lagging more commonly used in mines. This is referred to as support lagging and is shown on page 273.

Commercial steel roof bolt mats, shown on page 273 were especially designed for use in mines. They are commonly used with roof bolts to reduce spalling, and can be made to closely follow uneven surfaces.

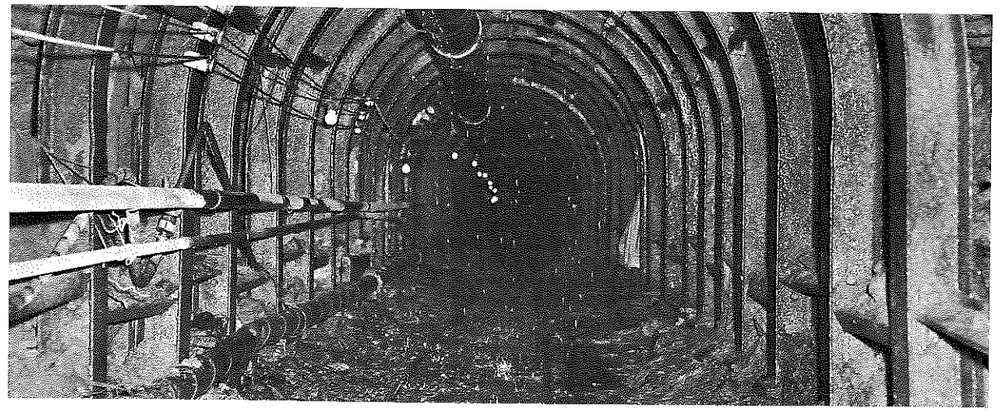
For temporary and quick roof support, Commercial furnishes their Hy-Jax, a hydraulic-yieldable and easily handled hand operated jack. These can be set up or removed by one man in just a few seconds. They are furnished in capacities of 10 and 22 tons in various lengths up to 105" (extended). Shown on page 273.

While material described in this section are classified as mine supports they may often be seen in use on tunnel construction projects and construction type material is frequently employed in mines. Knowledgable operators will use the materials best suited for their needs.

The following illustrations show applications of the various materials described in this section.



H-Beam rib with lagging on inside flange provides clean smooth interior.

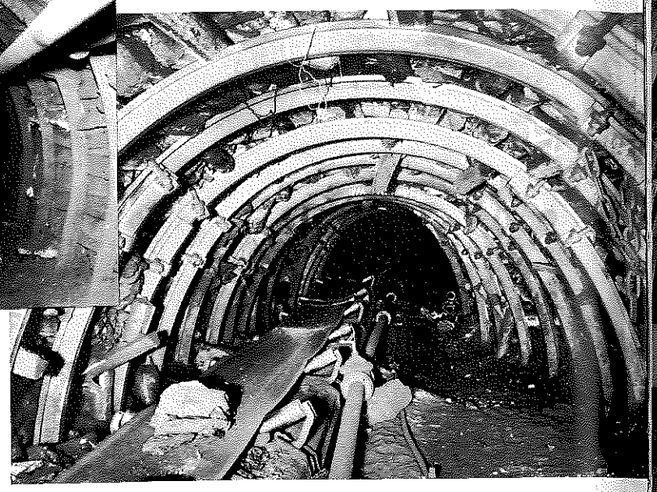


Arch Ribs in heavy ground. Blocking and lagging on back flange.

Complicated four way intersection and transition at bottom of a deep shaft.

YIELDABLE JOINT SUPPORTS

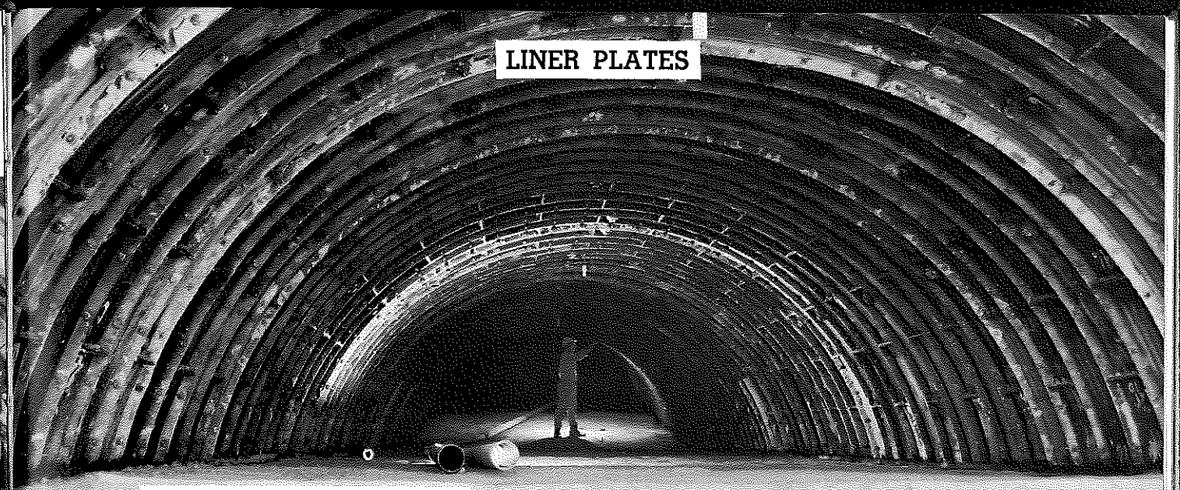
◀ 1. 4 piece set horseshoe shape.



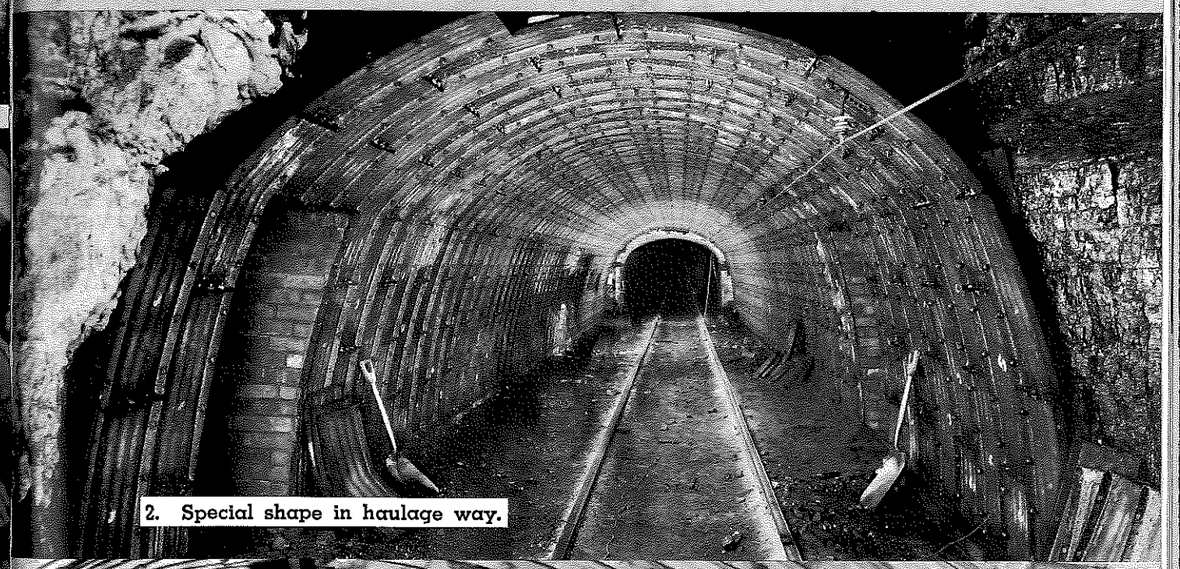
2. 3 piece set arch — note some sets deformed which evens out load. ▶

3. 4 piece set under heavy roof load. — Note crown bars for quick support.

LINER PLATES

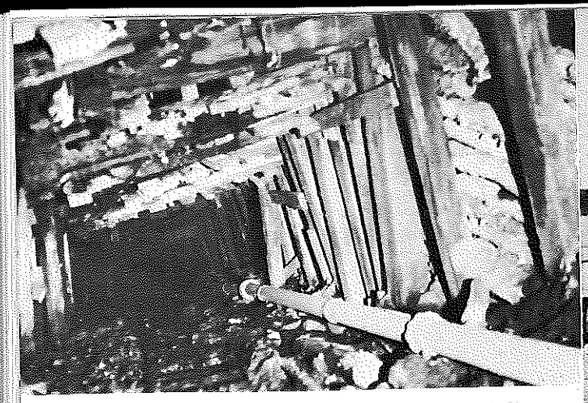


1. Portal support (Art. 300-307).

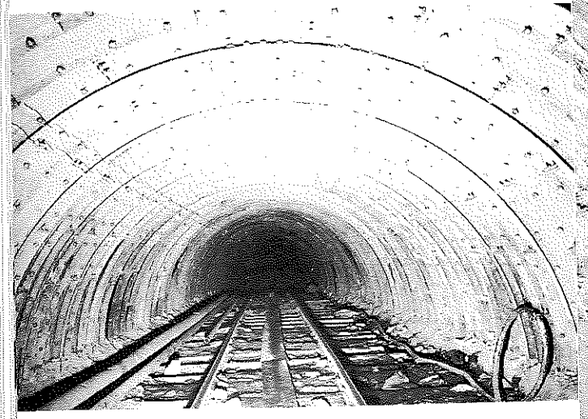


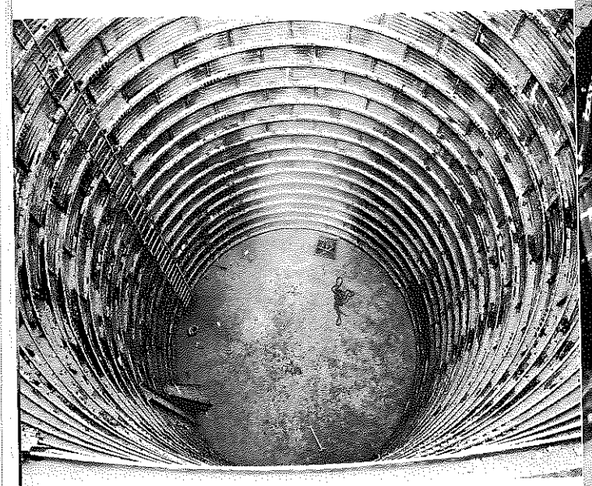
2. Special shape in haulage way.

3. Ventilation overcast.



▲ 4. Views of before and after Commercial's engineers worked on haulage slope. ▼

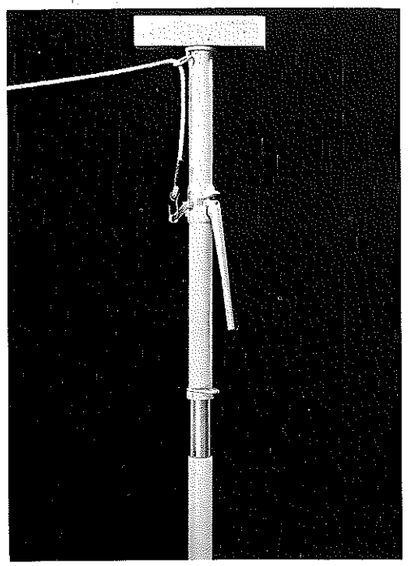




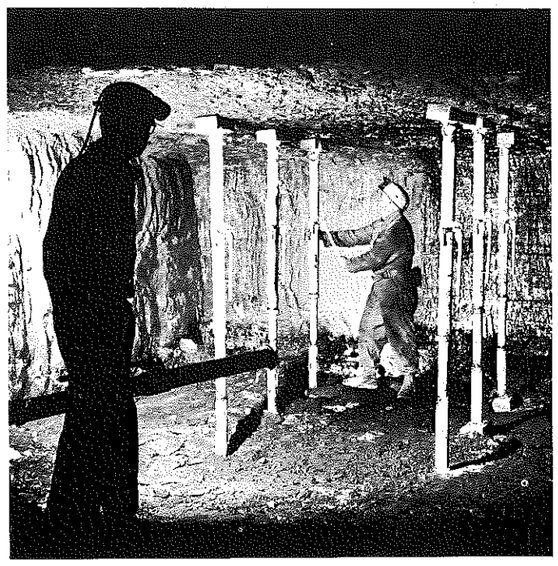
5. Shaft support.

6. Tunnel for recovery of coal or ore from stockpile.

HY-JAX



Closeup view of Hy-Jax.



Hy-Jax placed quickly and easily for prompt support of roof.

Note release lever which can be operated by hand line from remote location and HY-JAX safely recovered when no longer needed. Other types of roof support, including roof bolts, may be salvaged by using HY-JAX.

LAGGING

1. Steel lagging controls walls and roof.

2. Roof Bolt Mats adapt to surface.

Various designs of mats and lagging are available. For data in addition to illustrations shown write for folder.

Complete information relative to mine supports may be obtained from any of Commercial's representatives or by writing to the Main Office at Youngstown, Ohio.

Among others, these rock tunnels were built using steel supports from Commercial Shearing, Inc.:

ALASKA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
	1953	**Eklutna Tunnel**	Bureau of Reclamation	Polmer Const.	Flow

ARIZONA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Page	1958	**Glen Canyon Dam Tunnels** (Diversion, Spillway, Access.)	Bureau of Reclamation	Merritt-Chapman & Scott Co. Mountain States Const. Co. Frazier Davis Const. Co. Northwood, Inc.	Flow
Bisbee	1958	**Mule Pass Tunnel**	Arizona State Highway Dept.	Peter Kiewit Co.	1-Access
Yuma	1936 1937	***Gila Project** Main Canal Tunnels	Bureau of Reclamation	Mittry Bros. Const. Co.	Flow
Wenden	1965	**Alamo Dam & Appurtenant Works**	District Corps of Engineers	R. R. Hensler, Inc.	Flow
Ray	1971	**Kennecott Copper Corporation Water Diversion Tunnel**		Fluor-Utah Engineers & Constructors, Inc.	Flow

ARKANSAS

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Mountain Pine	1948	**Blakeley Mountain Dam** Power and Flood Control Tunnels	U.S. Army, Corps of Engineers	S. B. H. U. Co.'s	Flow and Pressure
Arkadelphia	1964	**DeGray Dam** Diversion Tunnel	U.S. Army, Corps of Engineers	Winston-Green Const. Co.	Flow
Winslow	1967	**Winslow Tunnel**	St. Louis-San Francisco Railway Co.	Morrison-Knudsen Co.	Railroad

CALIFORNIA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Livermore	1930	**Hetch-Hetchy Tunnel**	City of San Francisco	Force Account	Flow
Los Angeles County	1931	**Owens River Project** *Power Tunnel Repair	City of Los Angeles, Bureau of Water and Power	Force Account	Flow
Lone Pine	1949	**Owens River Gorge Tunnels** Power Tunnels		Owens Tunnel Contractor	Flow
Leevining	1934 1939	**Mono Basin Project** *Mono Craters Tunnel		Force Account	Flow

*Tunnels so marked are illustrated in this book.

CALIFORNIA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Leevining	1940	Leevining and Grant Lake Inlet Tunnels		A. Teichert & Sons, Inc.	Flow
	1933	**Los Angeles-Colorado River**	Metropolitan Water		Flow
	1939	**Aqueduct Tunnels**	District of So. California		
		*Iron Mountain, East		Winston Bros.	
		Iron Mountain, West		Utah Const. Co.	
		Coxcomb		Winston Bros.	
		*Eagle Mountain, West		L. E. Dixon & Bent Bros, Inc.	
		Eagle Mountain, East		Broderick & Gordon	
		Mecca Pass		Morrison-Knudsen Co., Inc.	
		Yellow Canyon		Force Account	
		Fargo Canyon		Force Account	
		Berdoo Canyon		Force Account	
		Pushawalla	NOTE: Steel Support was procured by the District and issued to the Contractors as required.	Force Account	
		7 Palms		Force Account	
		1000 Palms No. 1		Force Account	
		*Wide Canyon		Force Account	
		Long Canyon		Force Account	
		Blind Canyon		Force Account	
		Little Morongo		Force Account	
		Big Morongo		Force Account	
		Whitewater		West Const. Co.	
		*San Jacinto		Force Account	
		Bernasconi		Hamilton & Gleason Co.	
		Valverde		Dravo Const. Co.	
		Monrovia		West Const. Co., L. E. Dixon, Bent Bros., Inc. & Johnson Inc.	
		*Sierra Madre		J. F. Shea Co., Inc.	
		Pasadena		L. E. Dixon, Bent Bros., Inc., & Johnson, Inc.	
		San Rafael		L. E. Dixon, Bent Bros., Inc., & Johnson, Inc.	
		Ascot		J. F. Shea Co., Inc.	
		San Gabriel Spillway Hollywood		J. F. Shea Co., Inc.	
Santa Barbara National Forest	1933	**Maricopa-Ventura Highway Tunnels**	County of Los Angeles, Road Department	Angus MacDonnell	Highway
Los Angeles County	1937	**Angeles Forest Highway Tunnel**		Force Account	Highway
	1948	Highway Tunnel (Standard)		Force Account	Highway
	1950	Highway Tunnel (Standard)		Force Account	Highway

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CALIFORNIA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Los Angeles	1935	**Figueroa St. Tunnels**	California State Highway Dept.	L. E. Dixon Co., Bent Bros., Inc., & Johnson, Inc.	Highway
Plumas County	1947	**Feather River Project** West Point and Electric Tunnels	Pacific Gas & Electric Co.	Utah Const. Co. & Walsh Const. Co.	Flow
	1948	Rock Creek and Cresta Sections		Arundel Corp. & L. E. Dixon Co.	Flow
San Pablo	1949	**Oil Tank Farm**	Standard Oil Co.	Utah Const. Co.	Underpass
Goleta	1950	**Santa Barbara Project** Tecolote Tunnel	Bureau of Reclamation	Halvorson Contractors	Flow
Santa Maria	1950	**Cachuma Project** Cachuma Dam Outlet Tunnel	Bureau of Reclamation	Mittry Constructors	Diversion
Isabella	1949	**Isabella Project—Kern River** Main Dam, Diversion Tunnel	U.S. Army, Corps of Engineers	Walsh Const. Co.	Flow
	1958	**Colorado River Aqueduct**	Met. Water Dist. of So. California	Morrison-Knudsen, Macco	Water
Shasta	1958	**Trinity Dam Tunnels**	Bureau of Reclamation	Guy F. Atkinson Co.	Flow
Oroville	1957	**Poe Tunnel**	P. G. & E.	Utah Const.-Bates & Rogers	Flow
	1956	**Tri-Dam Project**	Oakland Irrigation District	Tri-Dam Constructors	Flow
Oroville	1963	**Oroville Dam Tunnels**	State of California	Frazier Davis Const.	Power & Div.
Oroville	1962	**Feather River Project**	State of California	Frazier Davis Const.	Power & Div.
O'Brien	1962	**Hazelview Summit Tunnel**	State of California	Grafe-Callahan Const.	Highway
Redding	1960	**Spring Creek Tunnel**	Bureau of Reclamation	Winston-Green-Drake	Flow
Redding	1960	**Clear Creek Tunnel**	Bureau of Reclamation	Shea-Morrison & Kaiser	Flow
Oroville	1959	**Feather River Tunnels 4 & 5**	Western Pacific R.R.	Peter Kiewit Co.	Railroad
Placerville	1959	**Camino Tunnel**		Walsh Const.	
Placerville	1959	**Jaybird Tunnel**	Sacramento Mun. Utility District	Frazier-Davis Co.	
San Francisco	1952	**Broadway Tunnel**	San Francisco Dept. of Public Works	Morrison-Knudsen Co., Inc.	Highway
Los Banas	1964	**Los Banas Creek Detention Dam**	Bureau of Reclamation	Guy F. Atkinson, Co.	Flow
Sacramento	1964	**White Rock Tunnel**	Sacramento MUD	Gates & Fox, Inc. Walsh Const. Co.	Flow
San Francisco	1965	**Berkeley Hills Tunnel**	City of San Francisco	Shea-Kaiser-Macco	Subway
Hobart Mills	1966	**Stampede Dam**	U.S. Bureau of Reclamation Department of Interior	R. A. Heintz Const. & Ray Kizer Const. Co. J. V.	Flow
Saugus	1968	**Saugus & Placerita Tunnels**	Metropolitan Water District of So. Calif.	Delaware V & M	Flow

*Tunnels so marked are illustrated in this book.

CALIFORNIA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Los Angeles	1968	**Majave River Fork Dam**	District Corps of Engineers U.S. Army	Kasler-Ball & Yeager	Flow
Auburn	1968	**Auburn Dam Exploratory Tunnels**	Bureau of Reclamation	Emil Anderson Constr. Co.	Pilot
Sylmar	1968	**Balboa Outlet Tunnel**	Metropolitan Water District of So. Calif.	R. A. Wattson-Winston Bros. J. V.	Flow
Castaic	1966	***Castaic Dam**	Calif. State Dept. of Water Resources Sacramento	Peter Kiewit	Flow
Livermore	1966	***Del Valle Dam**	Calif. State Dept. of Water Resources Sacramento	Green Winston Const.	Flow
Newhall	1966	**Newhall and Balboa Inlet Tunnels**	Metropolitan Water District of So. Calif.	Dixon-Arundel, MacDonald-Kruse-Kiewit	Flow
Bakersfield	1967	**Tehachapi Pumping Plant Discharge Lines**	Department of Water Resources	Perini-M & K—Brown & Root	Flow
Glendora	1967	**Glendora Tunnel Spec. 702**	Metropolitan Water District of So. Calif.	J. F. Shea Company	Flow
San Bernardino	1967	**San Bernardino Tunnel Spec. #67-39**	State of California The Resources Agency of Water Resources	J. F. Shea Company	Flow
Castaic	1966	**Angeles Tunnel—Aqueduct W. Branch Div. Spec. #66-22**	Department of Water Resources	Shea-Kaiser-Lockheed-Healy	Flow
San Francisco	1966	**Crystal Springs Bypass Tunnel**	San Francisco Water Department	Gates & Fox, Ball and Granite	Flow
Oakland	1968	**Bay Area Rapid Transit Project**			
		Broadway Tunnel	Bay Area Rapid Transit	J. F. Shea Company	Subway
San Francisco		Mission Street Tunnel		S & M Constructors	
Auburn	1971	**Auburn Dam Diversion Tunnel**	Bureau of Reclamation	Guy F. Atkinson Co.	Diversion

COLORADO

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
East Portal	1928	**Moffat Tunnel Pioneer Bore**	Moffat Tunnel Commission	Hitchcock & Tinkler	Flow
Empire	1937 1939	***William Fork Tunnel**	City and County of Denver	Broderick & Gordon	Flow
Denver	1950	**St. Louis Collection Conduit** Montezuma & St. Louis Tunnels	City and County of Denver	Board of Water Commissioners	Water
Granby	1942	**Colorado Big Thompson Project** *Granby Dam Diversion Tunnel	Bureau of Reclamation	Platt Rogers, Inc.	Pressure
Granby	1940	*Adams Tunnel, West Portal	Bureau of Reclamation	Platt Rogers, Inc.	Flow
Granby	1941	Adams Tunnel, West Portal	Bureau of Reclamation	Stiers Bros. Const. Co.	Flow
Lyons	1946	Prospect Tunnel	Bureau of Reclamation	Lowdermilk Bros.	Flow

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COLORADO

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Loveland	1947	Horsetooth Feeder Canal Tunnels 2, 3, 4, 5	Bureau of Reclamation	Tunnel Constructors	Flow
Loveland	1949	Horsetooth Supply Conduit Tunnel	Bureau of Reclamation	G. L. Tarlton Cont. Co.	Flow
Loveland	1949	Horsetooth Feeder Canal Siphon Tunnel	Bureau of Reclamation	Western Paving Cont. Co.	Pressure
Estes Park	1950	Olympus & Pole Hill Tunnels	Bureau of Reclamation	Wunderlich Contracting Co.	Flow
Estes Park	1950	Carter Lake Pressure Tunnel	Bureau of Reclamation	K. S. Mittry Const. Co.	Pressure
Loveland	1950	Carter Lake Reservior Outlet Tunnel	Bureau of Reclamation	Winston Bros. Co.	Flow
Loveland	1950	Rattlesnake Tunnel	Bureau of Reclamation	Gibbons & Reed Co.	Flow
Loveland	1951	Bald Mountain Pressure Tunnel	Bureau of Reclamation	Winston Bros. Co.	Pressure
Plataro	1949	**San Luis Project** Plataro Dam	Bureau of Reclamation	Hinman Bros. Const. Co.	Diversion
Tennessee Pass	1944	**Tennessee Pass Tunnel**	Denver & Rio Grande R.R.	Winston Bros.	Railroad
Utah Junction	1947 1948 1950	**Tunnel Relining**	Denver & Salt Lake Railway	Force Account	Railroad
Leadville	1950	**Leadville Drainage Tunnel**	Bureau of Mines	Utah Const. Co.	Drainage
Colorado Springs	1961	**Norad**	U.S. Army, Corps. of Engineers U.S. Airforce	Utah Const. Co.	Communication Center
Dillon	1960	**Harold D. Roberts Tunnel**	County of Denver	Blue River Constructors	Water
Dillon	1960	**Dillon Dam Tunnels**		R. B. Potashnick	Flow
Glenwood Springs	1964	**Glenwood Springs Tunnel**	Dept. of Highways State of Colorado	Colorado Constructors Inc.	Vehicular
Leadville	1964	**Homestake Dam—Missouri Creek Tunnel**	Cities of Aurora and Colorado Springs	Boyles Bros. Drilling Co.	Flow
Buena Vista	1964	**Homestake Tunnels Little Annie & Goddard Tunnels**	Cities of Aurora and Colorado Springs	Gibson & Roberts	Flow
Red Cliff	1964	**Homestake Tunnel Project Lake & Pitkin Counties**	Cities of Aurora and Colorado Springs	W. A. Smith Contracting Co. Quad Construction Co. Inc.	Flow
Colorado Springs	1964	**Contract #2 Rampart Tunnel**	City of Colorado Springs	F. H. Linneman	Flow
Loveland Pass	1964	**Straight Creek**	State of Colorado	Mid-Valley, Inc.	Pilot
Longmont	1967	**Button Rock Dam**	Bureau of Reclamation	Colorado-Horner	Flow
Chroma	1967	**O.S.O. Tunnel**	U.S. Bureau of Reclamation	O.S.O. Contractors	Flow
Marysville	1966	**Bullards Bar Dam**	Yuba County Water Agency	Perini-Yuba Associates	Flow
Leadville	1967	**Divide Tunnel**	Bureau of Reclamation	W. A. Smith & Quad Construction Co.	Flow

*Tunnels so marked are illustrated in this book.

COLORADO

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Basalt	1967	**Fryingpan**	Bureau of Reclamation	Winston, Foley, Hurley and Frazier Davis	Flow
Georgetown	1967	***Straight Creek Tunnel**	State of Colorado	Straight Creek Constructors	Vehicular
	1968	**Straight Creek Tunnel**			Highway Pilot
Georgetown	1967	**Eisenhower Tunnel Project**	State of Colorado	Straight Creek Constructors	Pilot Tunnel
	1968	Tunnel No. 1	Colorado Dept. of Highways	Al Johnson Construction, Gibbons & Reed, Western Paving Corp. Kemper Construction Co.	Vehicular
	1976	Tunnel No. 2		Peter Kiewit & Sons, Brown & Root, Inc.	
Basalt	1969	**Nast Tunnel**	Bureau of Reclamation	Peter Kiewit & Sons	Flow
Parshall	1970	**Henderson Project** Ore Haulage Tunnel	American Metals Climax	Dravo Corporation	Mine Entry

CONNECTICUT

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
New Haven	1948	**Wilbur Cross Parkway** West Rock Tunnel	State of Connecticut	Gull Contracting Co., Inc. & L. G. Defelice & Son., Inc.	Highway

DISTRICT OF COLUMBIA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
		WMATA Subway System	Washington Metor		
Washington	1970	Contract A-2		American Structures	Subway
		Contract A-4a		S. A. Healy Co.	
	1971	Contract A-4b		Granite Construction Co.	
	1973	Contract A-6a		Morrison-Knudsen & Asscciates	
	1975	Contract A-9a			
		Contract A-10a		James McHugh Construction Co.	
		Contract A-11a		J. F. Shea Company	
	1972	Contract C-4		Shea-Ball-S & M	
		Contract C-5		Shea-Ball	
	1974	Contract C-7		Ball-Healy	
	1972	Contract D-6		Healy-Kruse	
	1973	Contract D-4a		S & M-Traylor Bros.	
	1974	Contract D-8		Granite-Groves	
		Contract D-9		Fruin-Colnon Corp.	
	1973	Contract F-1b		Dravo Corp.	
	1976	Contract G-2		Healy-Ball-Greenfield	
	1974	Contract K-1		S & M Constructors	

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GEORGIA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
		Flint River Diversion Project	Atlanta Airport Authority		
Atlanta	1971	*Contract No. 10		S & M Constructors	Flow
	1975	*Contract No. 13		Grow Tunneling Co.	

HAWAII

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Honolulu	1956	**Kalihi Tunnels**	City of Honolulu	E. E. Black Ltd.	Highway
	1957	**Pali Road Tunnels**	Hawaii Highway Dept.	Tanaka	Highway
Pearl Harbor	1941	**Underground Fuel & Storage** Contract No. 4173	U. S. Navy	Hawaiian Pacific Naval Air Base Contractors	Storage

IDAHO

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Emmett	1936	**Boise Project** Black Canyon Canal, Payette Division, Six Tunnels	Bureau of Reclamation	J. A. Terteling & Sons	Flow
Trude	1936	**Upper Snake River Project** Island Park Dam Diversion Tunnel		Max J. Kuney Co.	Flow
Mountain Home	1941	***Anderson Ranch Dam** Diversion Tunnel		Morrison, Shea, Twits, Winston	Flow
Preston	1948	**Preston Bench Project** Station Creek Tunnel		Thatcher Const. Co., Inc..	Flow
Cambridge	1964	**Hells Canyon Hydroelectric Development**	Idaho Power Company	Peter Kiewit & Sons	Flow
Ahsahka	1967	**Dworshak Dam**	Bureau of Reclamation	Dworshak Dam Constructors	Flow

ILLINOIS

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Chicago		**Water Intake Tunnel** E. 79th Street from Lake Michigan West		S. A. Healy Co.	Flow
Chicago		**Water Intake Tunnel** E. 79th Street and Kedzie		Santucci Company	Flow
		Shafts for Deep Storm Drains	Metropolitan Sanitary District		
Chicago	1976	Addison to Wilmette		Kenny-Paschen-S & M	Storage
		Nashville Avenue		Kenny-Jaydee-S & M	
		O'Hare Water Reclamation Plant		Paschen-Newburg	

INDIANA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Mill Creek	1948	**Cagles Mill Reservoir** Outlet Tunnel	U.S. Army, Corps of Engineers	Kenny Const. Co.	Flow

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KANSAS

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Langley	1940	**Kanopolis Dam** Outlet Tunnel	U.S. Army, Corps of Engineers	Morrison-Knudsen Co., Inc.	Flow

KENTUCKY

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Edgoten	1947	**Camp Campbell Depot**	U.S. Army, Corps of Engineers	Morrison-Knudsen Co., Inc.	Storage
Jenkins	1947	**Jenkins Tunnel**	C & O Railway Co.	Haley, Chisholm & Morris	Railroad
Van Lear	1946	**Dewey Reservoir Project** Dewey Dam, Diversion Tunnel	U.S. Army, Corps of Engineers	Ryan Const. Corp.	Flow
Big Sandy River	1964	**Fishtrap Reservoir**	U.S. Army, Corps of Engineers	Cowin & Company, Inc.	Flow
Louisville	1967	**Cochran Hills**	Department of Highways*	Greer Bros. & Young Inc.	Vehicular
Pike County	1968	**Norfolk & Western Railroad Co.**	Norfolk & Western R.R. Company	Ohio Atlas Construction Company	Railroad
Baldrock	1966	**Laurel River Reservoir Project**	Corps. of Engineers	Fenix & Scisson	Flow

MARYLAND

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Baltimore	1937	**Montebello Tunnel**	City of Baltimore	J. F. Shea Co., Inc.	Flow
Baltimore	1947	**Patapsco Development** Liberty Rd.-Montebello Tunnel	City of Baltimore	Samuel R. Rosoff, Ltd.	Pressure
	1948	Patapsco-Liberty Rd. Tunnel		Samuel R. Rosoff, Ltd.	Pressure
	1946	**Liberty Heights-Patapsco Tunnel**		Stiers Bros. Const. Co.	Pressure
		Baltimore Subway System	Maryland Transit Administration		
Baltimore	1977	Bolton Hill		Fruin-Colnon-Horn Construction	Subway
		*Mondawmin Tunnels		Clevecon-Roger Au-Delaware Vianini	

MASSACHUSETTS

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Bondsville	1931	**Swift River Reservoir** Outlet Tunnel	Metropolitan District Water Supply Com.	Northern States Contracting Co.	Pressure
Southborough	1939	**Southborough Tunnel**		West Const. Co.	Pressure
Boston	1948	**Hultman Aqueduct City Tunnel Section** Contract #115	Commonwealth of Massachusetts Metropolitan Dist. Commission	Perini, Maney, Walsh & Rugo Const. Co.	Pressure
Auburn	1957	**Wochester Dam**	Corps of Engineers	Kero-Curly	Flow

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MICHIGAN

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
		Detroit Interceptor System	Detroit Metro Water Board		
Detroit	1970	Contract PCI-215		Demambro Construction	Sewerage
		Contract PCI-4		Corridor Constructors	
		Contract PCI-5 and 6			
	1971	Contract PCI-12A		Traylor-Mancinni & S & M	
	1972	Contract PCI-14		Traylor Bros. Co.	
		Oakland County Interceptor System	Oakland County Drain Commission		
Pontiac	1970	Augusta Drain		Greenfield Construction Company	
	1971	Joslyn Drain			

MINNESOTA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
St. Paul	1973	**St. Anthony Storm Sewer**	City of St. Paul	Acton, Inc.	Sewerage

MONTANA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Fort Peck	1936	**Fort Peck Dam** Diversion Tunnels	U.S. Army, Corps of Engineers	Silas Mason, Inc. & Walsh Const. Co., Force Account	Pressure
Big Horn County	1937	**Tonque River Dam** Diversion Tunnel	Montana State Water Conservation Board	Jerome C. Boespflug	Flow
Blassberg	1949	**Mullan Tunnel** Enlargement & Repair	Northern Pacific Railway	J. A. Terteling & Sons, Inc.	Railroad
Tiber	1968	**Tiber Dam**	Bureau of Reclamation	Foley Bros. Inc.	Flow
Trego	1966	**Libby Dam Project**	U. S. Corps of Engineers Seattle, Washington	Walsh-Groves	Railroad Relocation
Hardin	1962	**Yellowtail Dam**	Bureau of Reclamation	Morrison-Knudsen, Co., Inc.	Flow

NEVADA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Mercury	1959		Atomic Energy Commission	Reynolds Electric	
Henderson	1968	**River Mt. Tunnel**	Bureau of Reclamation	Utah Construction & Mining Co.	Flow
Elko	1972	**Carlin Canyon Highway Tunnel**	Nevada State Highway Dept.	Lockheed Construction Co.	Vehicular

NEW JERSEY

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Morristown	1941	**Filtration Gallery**	Town of Morrison	W.P.A.	Water

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NEW MEXICO

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Chama	1933	**El Vado Dam** Diversion Tunnel	Middle Rio Grande Conservancy Dist.	Force Account	Flow
Newkirk	1938	**Conchas Dam** Outlet Tunnel	U.S. Army, Corps of Engineers	McCarthy Improvement Co.	Pressure
Albuquerque	1947	**Project 76**	U.S. Army, Corps of Engineers	Peter Kiewit Sons' Co.	Storage
Los Alamos	1948	**Explosive Storage Tunnels** Unit A	Atomic Energy Commission	Brown & Root, Inc.	Storage
Chama	1964	**Azotea Tunnel**	U. S. Dept. of Interior Bureau of Reclamation	Azotea Contractors	Flow
Farmington	1965	**Navajo Indian Irrigation** Project Tunnel #1	Bureau of Reclamation	Fenix & Scisson	Flow
	1965	Project Tunnel #2	Bureau of Reclamation	Shea-Kaiser-Macco	
		Project Tunnels #3 and 3A		Fluor-Utah Engineers & Constructors, Inc.	
	1968	**Heron Dam**	Bureau of Reclamation	Colorado Constructors	Flow

NEW YORK

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
New York City	1930 1933	***City Water Tunnel No. 2**	Board of Water Supply, New York City	Patrick McGovern, Inc.	Pressure
	1937 1941	**Delaware Aqueduct Tunnels**	Board of Water Supply, New York City		
Valhalla		Contract 306 *Kensico By-Pass Kensico-Hill View		Associated Contractors, Inc.	
White Plains		Contract 307 *Kensico-Hill View *West Branch-Kensico		S. A. Healy Co.	
Plattekill		Contract 316 *Rondout-West Branch		Walsh Const. Co.	
Beacon		Contract 318 Rondout-West Branch		Pleasantville Constructors, Inc.	
Katonah		Contract 323 West Branch-Kensico		Seaboard Const. Corporation	
Mt. Kisco		Contract 324 *Kensico-West Branch		S. A. Healy Co. Frazier-Davis Const. Co. Subcontractor	
Mahopac		Contract 334 Rondout-West Branch West Branch-Kensico		W. E. Callahan Const. Co. and J. P. Shirely	
Ellenville		Contract 339 Lackawack Dam, Diversion		B. Perini & Sons, Inc.	
Liberty		Contract 360 Neversink Dam, Diversion		Geo. M. Brewster & Sons Co.	

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NEW YORK

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Downsville	1948	Contract 400 Downsville Dam		Walsh Const. Co. & B. Perini & Sons Co., Inc.	Diversion
Roscoe	1949	Contract 405, 406, 407 & 408 East Delaware Tunnel		Walsh Const. Co. & B. Perini & Sons Co., Inc.	Pressure
Neversink	1949	Contract 386 Neversink Tunnel		Frazier-Davis Const. Co.	Pressure
Colchester	1951	Contract 401 Downsville Dam Release Water Chamber		Bianchi, Central, Munroe-Langstroth, Rugo	Pressure
New York City	1935 1937	**Lincoln Tunnel**	Port of New York Authority		Highway
		New Jersey Land Section		Underpinning & Foundation Company	
		New York Land Section		Cornell Contracting Company	
New York City	1938	***Ward's Island Intercepter**	New York City Dept. of Public Works	Arthur A. Johnson Corp. and Necaro Co., Inc.	Sewer
New York City	1937	**Queens Midtown Tunnel** Contract 4	New York City Tunnel Authority	Walsh Const. Co.	Highway
	1941 1946	**Brooklyn-Battery Tunnel** Contract BBT-6		Mason & Hanger Co., Inc.	Highway
Hornell	1946	**Southern New York Flood Control Project** Almond Dam, Diversion Tunnel	U.S. Army, Corps of Engineers	Carlo Bianchi & Co., Inc.	Diversion
Buffalo	1949	**North Bailey Sewer** Contract "C"	Buffalo Sewer Authority	Herbert F. Darling	Sewer
Niagara Falls	1950	**Drainage Tunnel**	Hooker Electric Chemical Co.	Herbert F. Darling	Sewer
Rochester	1950	**Water Supply Tunnel** From Lake Ontario	City of Rochester	National Const. Co. and The Angelo Marro Const. Co.	Water
New York City		**New York Subway Additions**			
	1964	Sixth Avenue Subway	New York City Transit Authority	MacLean-Grove-Shepherd	Subway
	1970	East River Undercrossing		Kiewit-Morrison-Slattery	
	1972	Contract 131-A Sections #2 and #3		Central Park Constructors	
	1975	Contract 131-A Section 4		GMG Constructors	
	1970	**Third City Tunnel**	Board of Water Supply	Water Tunnel Contractors	Flow
	1970	**Contract 2-A**	Bureau Water Pollution Control	Perini-B&R-Ball-P&M	Sewerage
	1972	**Contract 3**			
	1973	***Port Richmond "A"**		Peter Kiewit Sons' Co.	
	1973	**Port Richmond "B"**		Richmond Constructors	
	1976	***Oakwood Beach**		Oakwood Beach Constructors	

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NORTH CAROLINA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
	1962	**Blue Ridge Parkway 2T2**	Bureau of Public Roads	Blythe Brothers Co.	Highway
	1963	**Hayward County Tunnel**	North Carolina State Highway Commission	Cowin & Company	Highway
	1961	**Blue Ridge Parkway 2T1**	Bureau of Public Roads	Robertson Fowler	Highway
Haywood County	1964	***State Proj. 8-19457-1-40-1(14)4**	Haywood County, North Carolina	Cowin & Company	Vehicular

NORTH DAKOTA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Riverdale	1948	**Missouri River Project** **Garrison Dam & Reservoir** Test Tunnel	U.S. Army, Corps of Engineers	Garrison Builders, Inc.	Diversion-Test Tunnel
	1949	Diversion, Flood Control and Power Tunnels		S. A. Healy Co. & Material Service Corp.	Diversion, Power Control

OHIO

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Cleveland	1948	**Nottingham Intake Tunnel** Land Section, No. B	City of Cleveland Dept. of Public Utilities	Kalill Co.	Water
	1949	**Kingsbury Run Relief Sewer**	Dept. of Sewers	National Construction Co.	Sewer
Avon	1947	**Avon Plant Enlargement** Intake and Discharge Tunnels	Cleveland Electric Illuminating Co.	Square Const. Co.	Water
Cincinnati	1949	**Tennyson Station** Supply Conduit	City of Cincinnati	Foley Const. Co.	Water

OKLAHOMA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Gore	1949	**Arkansas River Watershed** Tenkiller Ferry Dam Outlet and Pennstock Tunnels	U.S. Army, Corps of Engineers	W. K. McIlyar Const. Co.	Water
Broken Bow	1965	**Broken Bow Power Intake**	U.S. Army, Corps of Engineers	Al Johnson Const. Co.	Flow

OREGON

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Baker	1937	**Burnt River Project** Unity Dam, Outlet Tunnel	Bureau of Reclamation	J. A. Terteling & Sons	Flow
Tule Lake	1940	**Klamath Project** Tule Lake Dam Diversion Tunnel		J. A. Terteling & Sons	Flow
Campbell	1948	**Campbell Tunnel** Relining	Union Pacific R.R. Co.	Force Account	Railroad
Eugene	1961	**Round Butte Hydroelectric Project**	Portland General Electric Company	Gates & Fox	Power & Diversion

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OREGON

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Portland	1959	**Bull Run Dam #2 Tunnels**	City of Portland	Gates & Fox	Flow
	1956	**Swift Diversion Tunnel**	Pacific Power & Light Co. of Portland	Guy F. Atkinson Co.	Flow
Foster	1964	**Green Peter Reservoir**	U.S. Army, Corps of Engineers	Paul Hardeman Co.	Flow
Blue River	1964	**Blue River Tunnel**	U.S. Army, Corps of Engineers	Radco-Ausum	Flow
Portland	1966	**Portsmouth Tunnel**	City of Portland	Drummond-Bronneck, Inc.	Sewerage
Portland	1966	**Vista Ridge Tunnel**	Oregon State Highway Department	Donald M. Drake & Winston	Vehicular

PENNSYLVANIA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Kittanning	1938	**Pittsburgh Flood Control Project** *Crooked Creek Dam Diversion Tunnel	U.S. Army, Corps of Engineers	Geo. M. Brewster & Sons, Inc.	Flow
Tionesta	1938	**Tionesta Dam** Diversion Tunnel		S. J. Groves & Sons Co. & Lundin Bros., General Contractor: Murphy & Richman Contractors, Inc., Subcontractor	Flow
Saltzburg	1946	**Conemaugh Reservoir** Relocation of Conemaugh Div. Penna. Railroad		Herman Holmes	Railroad Double Track
St. Marys	1948	**Clarion River** East Branch Dam, Outlet Tunnel		Hunkin-Conkey Const. Co. & Shofner, Gordan & Hinman	Pressure
(near)	1939 1940	**Pennsylvania Turnpike Tunnels**	Pennsylvania Turnpike Comm.		Highway
Breezewood		*Sideling Hill		Arundel Corp.	
Carlisle		*Blue Mountain and Kittatinny		Bates and Rogers Const. Co.	
Berlin		Allegheny		Guthrie, Marsch, Peterson Co.	
Somerset		Laurel Hill		Hunkin-Conkey Const. Co.	
Breezewood		Ray's Hill		Mason & Hanger Co.	
Fort Littleton		Tuscarora		B. Perini & Sons, Inc.	
Berlin	1963	**Allegheny (No. 2)**		Merritt-Chapman & Scott Corp.	Highway
Philadelphia	1947	**Main Relief Sewer** Fairmont Ave. & 23rd St.	City of Philadelphia	Robert Lombardi	Sewer
	1948	23rd St. & Girard College		Jos. Lombardi & Sons	Sewer
	1948	**Southwest Treatment Works** Central Schuylkill River Siphon		The Leo Butler Co.	Sewer
	1949	**Relief Sewer** Wingohocking St.		Jos. Lombardi & Sons	Sewer
	1949	**East Central Schuylkill Interceptor Sewer**		Square Const. Co.	Sewer

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PENNSYLVANIA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
	1950	**Cheltenham Ave. Interceptor Sewer**		Tacony Cont. Corp.	Sewer
	1950	**Southwest Main Gravity Interceptor Sewer**		Peter Kiewit Sons' Co. & Bates & Rogers Const. Corp.	Sewer
	1946	Cable Tunnel	Philadelphia Electric Co.	Dravo Corp.	Service
Pittsburgh	1948	**Penn-Lincoln Parkway** Squirrel Hill Tunnels	Commonwealth of Pennsylvania Dept. of Highways	B. Perini & Sons, Inc., Assoc.	Highway
Pittsburgh	1957 1958	**Interceptor Sewers**	Allegheny County Sanitary Authority		
		Contract #50		B. Perini & Sons, Inc.	Sewer
		Contracts #46 & 49		Dravo Corp.	Sewer
		Contracts 28, 31, 45, 51, 52, 53		Mole Constructors	Sewer
		Contract #47		Harrison Const. & Frazier Davis Const.	Sewer
Vandergrift	1950	**Water Line Tunnel**	Municipal Authority of Westmoreland County	National Const. Co.	Service
Blue Ridge Summit	1951	**Camp Richie Project**	U.S. Army, Corps of Engineers	S. A. Healy Co.	
	1956	**T. J. Evans Tunnel**	Penna. Turnpike Commission	Lipsett Inc.	Highway
Pittsburgh	1958	**Fort Pitt Tunnel**	Commonwealth of Penna.	Merritt-Chapman & Scott Corp.	Highway
Kettle Creek	1959	**Kettle Creek**	U.S. Army, Corps of Engineers	Geo. M. Brewster & Sons, Inc.	
Philadelphia	1967	**Sedgley Avenue Sewer Tunnel Work No. S3395-FBD**	City of Philadelphia Water Department	Driscoll Construction Company	Sewer
Fanettsburgh	1966	**Cont. 120-1 Tuscarora Tunnel**	Penna. Turnpike Commission	Peter Kiewit	Vehicular
Roxbury	1966	**Cont. 607-709-1 Blue Mt.**	Penna. Turnpike Commission	C. J. Langenfelder	Vehicular
Roxbury	1966	**Kittatinny Mountain**	Penna. Turnpike Commission	C. J. Langenfelder	Vehicular
Warren	1966	**Kinzua Dam**	The Cleveland Elec. Illuminating	Hunkin-Conkey	Flow

RHODE ISLAND

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Pawtucket	1949	**Blackstone Valley Interceptor Sewer** Section A	State of Rhode Island & Providence Plantations Blackstone Valley Sewer District Commission	Gil Wyner Co., Inc.	Sewer
	1950	Section B, Contract II		Marinucci Bros. & Co., Inc.	Sewer

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SOUTH CAROLINA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Seneca	1967	**Keowee-Toxaway Project**	Duke Power Company	Cowin & Company	Flow

SOUTH DAKOTA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Pickstown	1949	**Missouri River Project Fort Randall Dam** Diversion, Power and Control Tunnels	U.S. Army, Corps of Engineers	Silas Mason Co., Inc.	Flow Pressure
Pierre	1955	**Oahe Reservoir** Diversion and Power Tunnels	U.S. Army, Corps of Engineers	Mittry Constructors	Flow

TENNESSEE

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Etowah	1941	**Apalachia Tunnel**	Tennessee Valley Authority	Force Account	Flow
	1962	**Southern R.R. Tunnel**	Southern R.R. Company	Tecon & Co.	Railroad
	1962	**Southern R.R. Tunnel**	Southern R.R. Company	Cowin & Co.	Railroad
Chattanooga	1955	**Missonary Ridge Tunnel**	State Highway Dept.	Codell Const. Co.	Highway

TEXAS

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Austin	1973	**Crosstown Wastewater Interceptor 5029-3**	City of Austin	Granite Construction Company	Sewerage
Houston	1973	**Storm Sewer Extension** McCarty Street Area	City of Houston	John Holland Construction Co.	

UTAH

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Hyrum	1934	**Hyrum Dam** Diversion Tunnel	Bureau of Reclamation	J. A. Terteling & Sons	Flow
Spring City	1938	**San Pete Project** Spring City Tunnel		Dan Teters & Co., Inc.	Flow
		Provo River Project Utah-Salt Lake Aqueduct			Flow
Draper	1940	*Olmstead and Alpine-Draper Tunnels		Thompson-Markham Co.	Flow
Keely	1940	Duschesne Tunnel		Utah Const. Co.	Flow
Bingham Canyon	1937	*Highway Tunnel	Utah Copper Co.	Utah Const. Co.	Highway
Lark	1948	**Mascotte Tunnel**	Kennecott Copper Co.	Utah Const. Co.	Railroad

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UTAH

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Bingham	1951	**Bingham Canyon** "C" Line Tunnel		Utah Const. Co.	Railroad
Wanship	1955	**Wanship Dam** Diversion, Power Tunnel	Wanship-Bureau of Reclamation	Utah Const. Co.	Flow
Manila	1959	**Flaming Gorge** Diversion, Power Tunnel	Bureau of Reclamation	Coker Construction	Flow
Duchesne	1967	**Starvation Dam**	Bureau of Reclamation	Centennial Development	Flow
Bonneville Unit	1967	**Strawberry Aqueduct Water Hollow Tunnel**	Bureau of Reclamation	Boyles Bros.	Flow
Duchesne	1976	VAT Tunnel	Bureau of Reclamation	J. F. Shea Company	Flow

VERMONT

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Thetford	1946	**Union Village Dam** Diversion Tunnel	U.S. Army, Corps of Engineers	Savin Const. Co.	Water

VIRGINIA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Jewell	1967	**Pocahontas Div. Pevler Tunnel**	Norfolk & Western Railway Company	Norfolk & Western R.R. Co.	Railroad
Carnot	1968	***Big Walker Tunnel**	Commonwealth of Virginia Dept. of Highways	Langenfelder, Raymond	Vehicular

WASHINGTON

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Yakima		**Yakima Project** Yakima Ridge Canal Roza Division	Bureau of Reclamation		Flow
	1936	*Tunnels Nos. 1, 2, 3		Morrison-Knudsen Co.	
	1939	*Tunnels Nos. 5, 7, 8		J. A. Terteling & Sons	
Coulee City	1947	**Columbia Basin Project** Bacon Canal Tunnel		T. E. Connolly, Inc.	Flow
Seattle	1964	**Cont. 63-5 Lake City Tunnel**	Municipality of Metropolitan Seattle	Kemper-Ferrera	Sewerage
Coulee Dam	1968	**Grand Coulee Dam**	U. S. Dept. of Interior Bureau of Reclamation	Gibbons & Reed	Flow
Seattle	1967	**Elliot Bay Interceptor Sewage Disposal Project**	Municipality of Metropolitan Seattle	Traylor-Pamco	Sewerage
Near Monroe	1968	**Lake Chaplain Storage Reservoir Everett Water Tunnel**	City of Everett Snohomish City, Wash.	Ausum-Drummond-Bronneck	Flow
Azer	1967	**Lower Monumental Lock & Dam**	Seattle Dist. Corps of Engineers	Nelson Graven J.V.	Flow

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WASHINGTON

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Cascade	1976	***Bonneville Railroad Tunnel**	Corps of Engineers	Granite Construction Company	Railroad
Coulee City	1976	***Bacon Tunnel**	Bureau of Reclamation	Guy F. Atkinson Co.	Flow

WEST VIRGINIA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Gauley Bridge	1930	**Power Project**	New Kanahwa Power Co.	Reinhart & Dennis Co.	Flow
White Sulphur Springs	1930	**Tunnel Enlargement**	Chesapeake and Ohio R.R. Co.	Walton Const. Co.	Railroad Double Track
	1946	**Fort Spring Tunnel**	Chesapeake and Ohio R.R. Co.	Haley, Chisholm & Morris, Inc.	Railroad Double Track
Holden	1947	**Holden Tunnel**		Haley, Chisholm & Morris, Inc.	Railroad
Grafton	1936	**Tygart Dam R.R. Relocation—** *Lane Tunnel *Knight's Tunnel	U.S. Army, Corps of Engineers	Guthrie, Marsch, Walker Co.	Railroad
Coaldale	1947	**Elkhorn Tunnel**	Norfolk & Western Railway Company	Haley, Chisholm & Morris, Inc.	Railroad Double Track
Berryburg	1947	**Berryburg Tunnel**	Baltimore & Ohio R.R. Co.	Bates & Rogers Constr. Corp.	Railroad
Clarksburg	1950	**Tunnel #1, Relocation**	Baltimore & Ohio R.R. Co.	Bates & Rogers Constr. Corp.	Railroad
Summersville	1961	**Summersville Reservoir** Diversion & Power Tunnels	U.S. Army, Corps of Engineers	J. A. Jones Construction Company	Flow
Wheeling	1963	**Wheeling Tunnel**	State of West Virginia	C. J. Langenfelder	Highway
Parkersburg	1963	**Baltimore & Ohio Railroad Tunnel**	Baltimore & Ohio Railroad Co.	C. J. Langenfelder	Railroad
Sutton	1956	**Little Elk Tunnel**	U.S. Army, Corps of Engineers	Agnew-Joseph Co.	Railroad
Wheeling	1964	**Wheeling Vehicular Tunnel**	State Road Commission	C. J. Langenfelder	Vehicular
Hanover	1968	**Norfolk & Western #2**	Norfolk & Western R.R. Company	Paramount Pacific Inc.	Railroad
Justice	1968	**Relocation for R. D. Bailey Lake Reservoir**	Norfolk & Western R.R. Company	Mountain States	Railroad

WYOMING

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Alcova	1936	***Kendrick Project** Casper Canal Tunnels	Bureau of Reclamation	W. E. Callahan Const. Co. and Gunther & Shirley	Flow

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WYOMING

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Cody	1936	**Shoshone Project** Shoshone Canyon Conduit Tunnels		Utah Const. Co.	Flow
Riverton	1948	**Riverton Project** Wyoming Canal		Morrison-Knudsen Co., Inc. Wixon & Crowe (Subcont.)	Flow
Boysen	1949	**Wyoming, Missouri Basin Proj.** Boysen Dam Railroad Relocation		Morrison-Knudsen Co., Inc.	Railroad
Moorcroft	1950	**Missouri River Basin Project** Keyhole Dam, Outlet Tunnel		Gates & Fox Co.	Flow
Aspen	1947	**Aspen Tunnel**	Union Pacific R.R. Co.	Morrison-Knudsen Co., Inc.	Railroad
Casper	1958	**Fremont Canyon**	Bureau of Reclamation	Coker-Kiewit-Cunningham	Flow
Casper	1955	**Glendo Dam**	Bureau of Reclamation	Gates & Fox Inc.	Flow

BOLIVIA, SOUTH AMERICA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Cochabamba		**Corani Hydroelectric Tunnel**	Corporacion Boliviana de Fomento	J. A. Jones Const. Co.	Flow, Power

BRAZIL, SOUTH AMERICA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Paraiba	1949	**Paraiba Project Santa Cecilia & Vigario Tunnels**	Rio de Janeiro Tramway Light & Power Co.	Cia. Morrison-Knudsen Do Brazil, S.A.	Flow

CANADA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Niagara Falls	1952-3	**Sir Adam Beck G.S. #2**	Hydroelectric Power Comm. of Ontario	Perini-et, al. Raynor Atlas	Flow
Prov. of Saskatchewan		**Saskatchewan Dam Tunnels**	Prairie Farms Rehabilitation Assoc.		Flow
Isle Maligne Quebec	1959	**Pass Dangereuse** (Chute Des Passe)	Power Development Comm. Canada	Perini-McNamara Quemont	Power Adit Water
Garibaldi Br. Columbia	1956	**Cheakamus Tunnel**	Power Development Comm.	Mannix Stolte	Power & Diversion
Hudson Hope Br. Columbia	1962	**Peace River-Contract #3** Diversion Tunnels	British Columbia Electric Co. Ltd.	Portage Mountain Constructors	Diversion

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COLOMBIA, SOUTH AMERICA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
El Colegio	1963	***El Colegio Tunnel**	Empresa de Energia Electrica de Bogota	Morrison-Knudsen Co., Inc. Agts. for Constructora Emkay, S.A.	Flow, Power
Cali		***Calima Hydroelectric Tunnel**	Corporacion Autonoma Regional del Cauca	Compania Perini, S.A.	Flow, Power

FRANCE

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Sioule River	1964	***Chute de Besserve**	Elec de France	S.G.E. (Paris)	Diversion Power
Paris	1964	**Saint Placide Station**	R.A.T.P.	Compangnie Europeene de Travaux-CETRA S.A.	Railroad

GREECE

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Acheloos River	1962	***Kremasta Hydroelectric** Diversion, Power	Public Power Comm. of Greece	Kaiser Engineers and Constructors, Inc.	Water

LUXEMBOURG

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Kautenbach	1963	**Shieburg Tunnel**	Societe Nationale des Cheminis de Fer Luxembourgeois	Owner	Railroad

MEXICO

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Mexico City	1968	**Tunnel De Canal De Miramontes**	Departmento Distrito Federal	El Aguila S.A. De C.V.	Sewerage

PERU, SOUTH AMERICA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Carbonyacu		**Graton Tunnel** at Casapalca Mine	Cerro De Pasco Corp. of Peru	Cero Corp.	Transportation

SANTO DOMINGO

Santo Domingo	1975	**Sabana Yequa**	Dominican Republic	Atkinson International	Diversion

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SWITZERLAND

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Olten	1964	**Belchen Tunnel**	Government	Costruzioni Stradali E. Civili S.A.	Highway
Hagendorf	1964	**Belchen Tunnel**	Government	Costruzioni Stradali E. Civili S.A. Lugano	Highway
Personico	1964	**Impianto Nuova Biaschina**	Nuova Biaschina Kraftwerk	Consorzio Val d'ambra	Hydro-electric
Lugano	1965	**Lugano Vedeggio Tunnels**	Communal Electric Works	M. & F. Muttoni Ing.	Hydro-electric
Nivo	1965	**Impianto Nuova Biaschina**	Nuova Biaschina Kraftwerk	E. Baumann A.G. Consorzio Galleria Nivo	Hydro-electric
Engadiner Valley	1965	**Fensterstollen** Val Flin	Engadiner Kraftwerk A.G.	Torno A.G.	Hydro-electric

WEST PAKISTAN

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Jhelum River Indus River Basin	1963	***Mangla Dam Tunnels** Diversion, Power	WAPDA	Guy F. Atkinson	Water

VENEZUELA, SOUTH AMERICA

LOCATION	YEAR	PROJECT	OWNER	CONTRACTOR	TYPE OF TUNNEL
Bet. LaGuaira & Caracas	1953	**Boqueron Tunnel**	Government	C. A. Morrison-Knudsen	Highway

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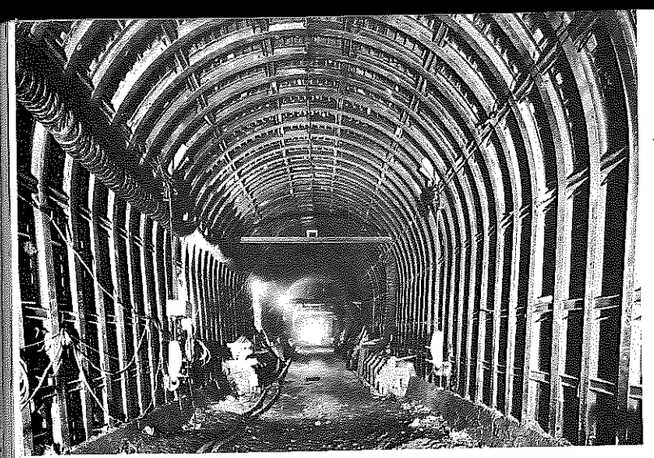
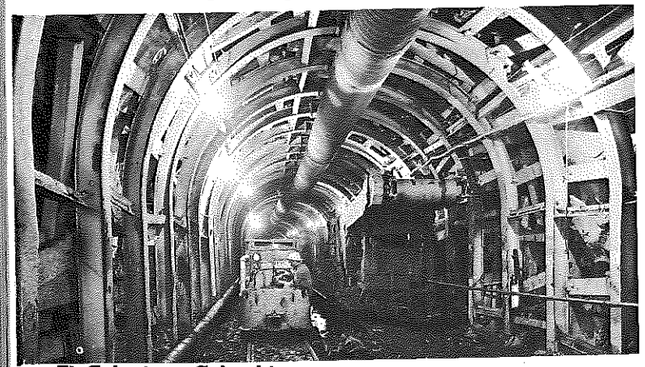
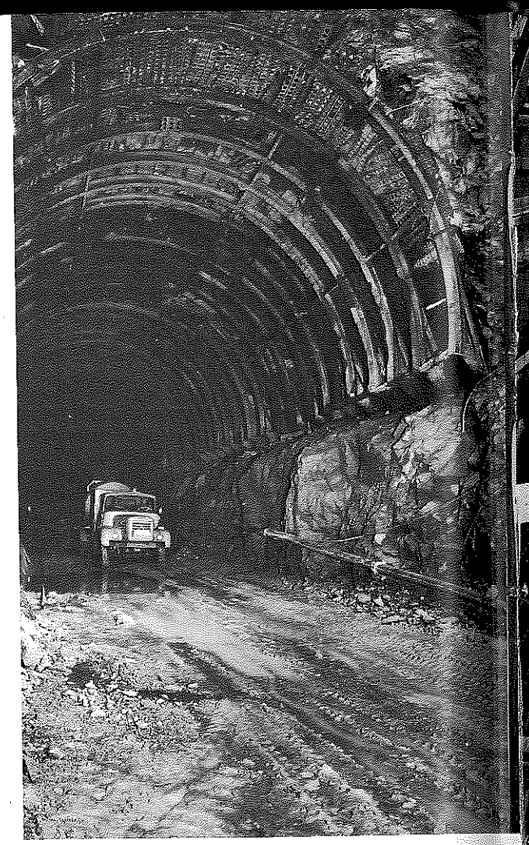


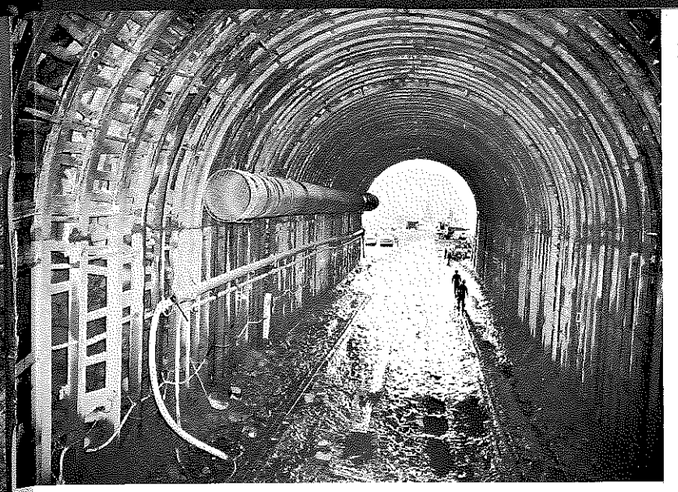
Milano Subway — Italy



El Colegio — Colombia

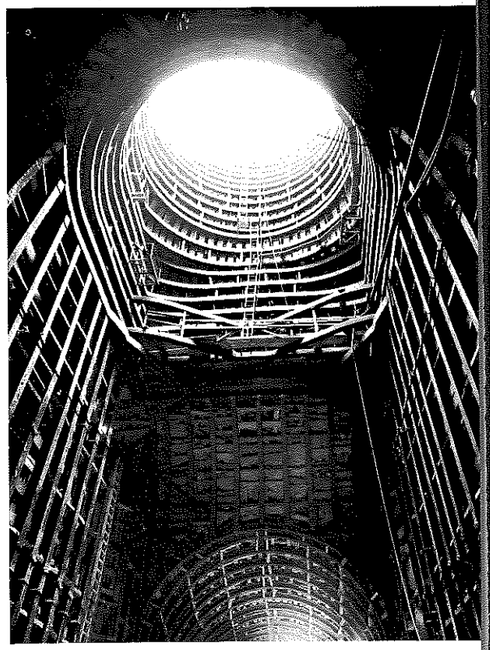


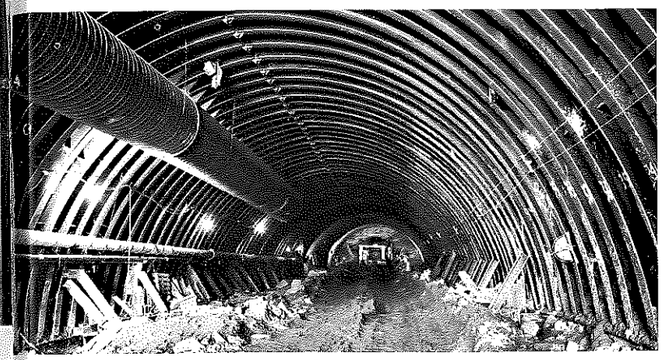
Chute de Besserve — France



Big Walker, Vehicular—Virginia

Del Valle, Dam—California



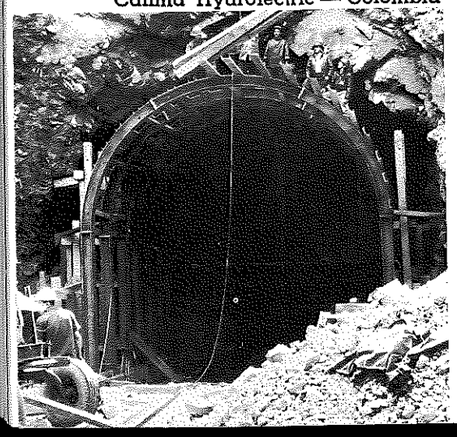


East River Mountain, Vehicular—Virginia

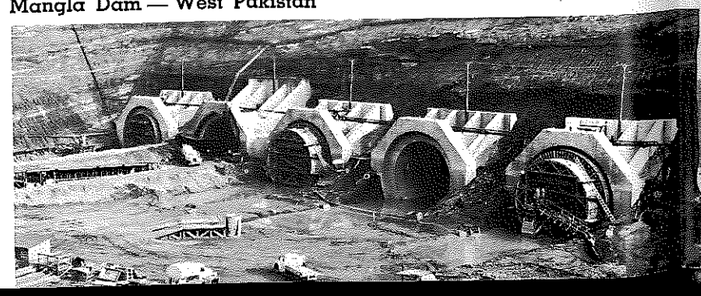
Eisenhower Memorial, Vehicular—Colorado

Kremasta Dam — Greece

Calima Hydrolectric — Colombia



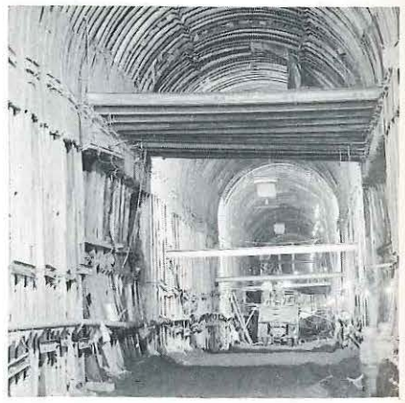
Mangla Dam — West Pakistan



Port Richmond, Interceptor—New York

Bacon Siphon, Water Tunnel—Washington

Oakwood Beach, Interceptor—New York



Bonneville, Railroad Tunnel—Washington